SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN, MALDEN, REVERE, AND SAUGUS
MASSACHUSETTS

HYDROLOGY AND HYDRAULICS

APPENDIX B

Hydrologic Engineering and
Hydraulics and Water Quality Sections
Water Control Branch
Engineering Division

Department of the Army
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Appendix (B) Includes descriptions of: the tidal hydrology and hydrology of interior runoff in the study area, and of wave runup and seawall overtopping, interior flood stage frequencies, tide levels, flushing, currents, and sea level rise effects without and with the selected project for various gated openings.

Appendix (C) Includes descriptions of existing water quality conditions in the estuary and explores potential changes associated with the selected plan.

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SAUGUS RIVER AND TRIBUTARIES FLOOD DAMAGE REDUCTION STUDY
Lynn, Malden, Revere and Saugus, Massachusetts/Summary of Study Reports:

Main Report and Environmental Impact Statement/Report (EIS/EIR): Summarizes the coastal flooding problems in the study area and alternative solutions; describes the selected plan and implementation responsibilities of the selected plan; and identifies environmental resources in the study area and potential impacts of alternative solutions, as required by the Federal (NEPA) and state (MEPA) environmental processes.

Plan Formulation (Appendix A): Provides detailed information on the coastal flooding problem and the alternatives investigated; includes: sensitivity analyses on floodgate selection (including location and size of gates and sea level rise); optimization of plans; comparison of alternative measures to reduce impacts; and public concerns.

Hydrology and Hydraulics (Appendix B): Includes descriptions of: the tidal hydrology and hydrology of interior runoff in the study area, and of wave runup and seawall overtopping, interior flood stage frequencies, tide levels, flushing, currents, and sea level rise effects without and with the selected project for various gated openings.

Water Quality (Appendix C): Includes descriptions of existing water quality conditions in the estuary and explores potential changes associated with the selected plan.

Design and Costs (Appendix D): Includes detailed descriptions, plans and profiles and design considerations of the selected plan; coastal analysis of the shorefront; detailed project costs; scope and costs of engineering and design; scope and costs of operation and maintenance; and design and construction schedules.

Geotechnical (Appendix E): Describes geotechnical and foundation conditions in the study area and the design of earth embankment structures in the selected plan.

Real Estate (Appendix F): Describes lands and damages, temporary and permanent easements and costs of the selected plan, including the five floodgate alignments studied.

Economics (Appendix G): Describes recurring and average annual damages and benefits in study area floodzones; economic analysis and optimization of alternative plans.

Socioeconomic (Appendix H): Describes the socioeconomic conditions in the study area and the affects of the selected plan on development in the floodplain and estuary.

Planning Correspondence (Appendix I): Includes all letters between community officials, agencies, organizations and the public and the Corps prior to agency and public review of the draft report.

EIS/EIR Comments and Responses (Appendix J): Includes all public comments and Corps responses to letters received during the public review of the draft report.

Environmental (Appendix K): Includes basic data from investigations of environmental resources in the study area and presents the Mitigation Incremental Analysis.

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN, MALDEN, REVERE, AND SAUGUS
MASSACHUSETTS

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HYDROLOGY AND HYDRAULICS

APPENDIX B

1. PURPOSE

This report presents hydrologic and hydraulic information and analysis pertinent to the planning and design of tidal flood control in the lower Saugus and Pines River estuary, and coastal vicinities in Lynn, Revere, and Saugus, Massachusetts. Since flood reduction in Malden is relatively small, no detailed hydrologic and hydraulic analysis was conducted for that area. Included are sections on watershed description, climatology, streamflow, estuary storage capacity, tidal hydrology, flood plain zones, flood level frequencies and hydrologic and hydraulic design criteria and analysis. A discussion of rising sea level effects is also included. Alternative structural plans for flood reduction involved a series of individual local protective measures (LPP option) as compared with a Regional Plan involving a tidal floodgate structure at the mouth of the Saugus River and tied into a system of dikes and walls along the shore. Further discussion of the alternative plans is presented in the Main Report. This report presents hydrologic information and analyses with and without flood reduction plans.

2. WATERSHED DESCRIPTION

The Saugus River, including its Pines River tributary, (shown on plate 1) has a total watershed area of approximately 47 square miles, covering portions of the towns of Reading, Wakefield, Lynnfield, Saugus, Lynn, Malden and Revere, Massachusetts. The Saugus River originates at the outlet of Lake Quannapowitt (EL 84 feet NGVD) in Wakefield and flows a meandering southeasterly course for 13 miles to its point of discharge into Boston Broad Sound, on the border between Lynn and Revere. The lower 4.7 miles of the Saugus River is a tidal estuary; where it enters tide water near Woodbury (Hamilton) Street crossing, its drainage area is about 25.7 square miles or 55 percent of the total watershed. The remaining lower 21 square miles is comprised of multiple storm drained coastal urban areas, extensive tidal marshland and the Pines River Tributary. From Lynn there is 7.2 square miles of urban area plus 1.8 square miles of Birch and Breeds Ponds watershed draining to the Saugus River estuary from the north. From the south there is 3.4 square miles of local watershed in Saugus draining directly to the Saugus estuary plus the Pines River tributary with its total watershed of about 8.9 square miles. The Pines River originates at the confluence of Linden and Town Line Brooks in Revere near the Revere-Malden town line. Drainage area at its origin is 4.0 square miles with an interim 4.9 square miles of local drainage. The Pines River flows northeasterly a distance of approximately 3 miles, joining the main stem of Saugus River about one-half mile upstream of its mouth at Broad Sound. The Pines River is a tidal estuary for its entire 3 mile length.

The Saugus River watershed is highly urbanized due to its close proximity to Metropolitan Boston and its major transportation corridors, notably: Highway Routes 107, 1, 1A, and 128 (I-95), plus the Massachusetts Bay Transportation Authority (MBTA) and Boston & Maine Commuter Rail Service. However, the river basin remains hydrologically "sluggish" due to its flat gradient and numerous small lakes and marshlands. Lakes and marshlands make up about 10 percent of the total watershed area. The Saugus River has a total fall of 80 feet in its 13-mile course to tide water and the Pines River gradient changes with the tide for its entire length. The basin does have some hilly uplands, controlled by frequently exposed bedrock, with elevations of nearly 300 feet NGVD. The overburden in the lower coastal area of the Saugus basin is predominantly an impervious silty clay. The lower southeastern border of the Saugus River watershed fronts on Broad Sound with its divide extending along about 6 miles of ocean shore front; about 4 miles in Revere and 2 miles in Lynn.

3. CLIMATOLOGY

a. General. The Saugus River Basin and its coastal vicinity, located at 42 degrees north latitude, has a cool, semi-humid variable climate, typical of New England. Its climate is somewhat less harsh than in the higher inland areas of New England due to the moderating effect of the adjacent ocean waters. However, fronting the coast exposes the area to coastal storms that move up the Atlantic coast often with accompanying intense rainfall, winds, and flood producing storm tides and waves. The mean annual temperature at Revere is 51° Fahrenheit with mean monthly temperatures varying from 72°F in July to 29°F in January and February. Extremes in temperature vary from summertime highs in the nineties to wintertime lows in the minus teens.

b. Precipitation. The mean annual precipitation in the Revere-Lynn area is 42 inches based on more than 100-years of continuous record at neighboring Boston. Precipitation is distributed quite uniformly throughout the year averaging about 3.5 inches per month. Short duration intense rainfall often accompanies fast moving frontal systems, thunderstorms and coastal storms. Also, a portion of the annual precipitation occurs as snowfall. Average annual snowfall at Boston is 43 inches occurring primarily during December through March. Data on snowpack is not available in the basin, but based on Corps of Engineers data at inland basins, it is estimated that maximum water equivalent occurs, on average, about the 1st of March, ranging from zero to about 6 inches, with an average of about 2.7 inches. Mean, maximum and minimum monthly precipitation recorded at Boston, MA is listed in table 1.

TABLE 1

MONTHLY PRECIPITATION
BOSTON, MASSACHUSETTS
Elevation 15 Feet NGVD
169-Years of Record
(Inches)

Month	Mean	Maximum	Minimum
January	3.69	10.55	0.35
February	3.45	9.98	0.45
March	3.95	11.75	Trace
April	3.80	10.83	0.20
May	3.42	13.38	0.25
June	3.22	13.20	0.27
July	3.29	12.38	0.52
August	3.77	17.09	0.37
September	3.29	11.95	0.21
October	3.38	8.89	0.06
November	3.99	11.63	0.59
December	3.79	9.74	0.26
Annual	42.98	67.72	23.71

Peak storm rainfall frequency-duration data, as reported in US Weather Bureau Technical Paper No. 40, is summarized in table 2. Some storm rainfalls accompanying recent abnormally high ocean tide events are listed in table 3. Although rainfall is frequently associated with high storm tide events, there appears to be little correlation between magnitude of storm tide and intensity of rainfall. The greatest known storm rainfall in the basin is believed that of 5 to 7 October 1962, when 48 to 72-hour rainfall amounts varied from 10 to 14 inches over the Saugus basin with one day totals of about 8 inches. This storm was apparently centered over the basin and produced the greatest known Saugus River floodflow. This rainfall was the combined result of a "Northeaster" coastal storm on October 5-6 and the fringe of tropical storm "Daisy" on October 6 and 7. However, ocean tides were not abnormally high during this major rainfall.

TABLE 2

RAINFALL - FREQUENCY - DURATION
USWB TECHNICAL PAPER 40
BOSTON, MASSACHUSETTS

Annual Frequency	Rainfall (Inches) Duration in Hours				
	1	2	6	12	24
50% (2-Yr. Freq.)	1.1	1.5	2.1	2.6	3.1
20% (5-Yr. Freq.)	1.5	2.0	2.8	3.4	4.0
10% (10-Yr. Freq.)	1.8	2.3	3.3	3.9	4.6
2% (50-Yr. Freq.)	2.4	3.1	4.3	5.1	6.0
1% (100-Yr. Freq.)	2.6	3.3	4.7	5.8	6.8
Standard Project Storm	3.5	4.8	9.0	10.6	12.4

TABLE 3

STORM RAINFALLS ACCOMPANYING
RECENT STORM TIDES ABOVE 9.0 FEET NGVD

Storm Event	7 Feb 1978	2 Jan 1987	25 Jan 1979	19 Feb 1972
Ocean Tide (ft, NGVD)	10.3	9.4	9.3	9.1
Tide Freq. Percent	1	6	7	9
Maximum Rainfall				
1 Hour (inches)	0.2*	0.4	0.4	0.5
6 Hour (inches)	1.0*	1.3	1.8	1.9
24 Hour (inches)	2.6*	2.2	2.7	2.5

* Water equivalent of snow

An earlier major rainfall occurred in August 1955 when about 7.3 inches of rainfall occurred over the basin in a 48-hour period. More recent storm events, typical of the basin, was that of January 1979, when 3.1 inches was experienced on the 21st followed by 2.1 inches on the 25th and as recent as April 1987 when 5.8 inches were experienced during a 3-day period.

4. RUNOFF

a. General. There are no streamflow gaging stations on the Saugus River, however, based on gaged streams in the region, it is believed that average annual runoff is about 23 inches or about 50 percent of average annual precipitation. Twenty-three inches of runoff converts to an average annual flow rate, per square mile of watershed, of about 1.7 cfs. Thus, the Saugus River, with a total watershed area of 47 square miles, would have an estimated average natural runoff of about 80 cfs. However, the average natural flow is reduced by an estimated 10 to 12 cfs by Wakefield and Lynn partial water supply sources in the basin. This water is supplied and used within the basin, but waste water is directed out of the basin, thereby reducing natural stream flow. Mean, maximum and minimum natural runoff in the basin, based on gaged runoff from the Parker River Basin at Byfield, Massachusetts, is listed in table 4.

TABLE 4

SAUGUS RIVER BASIN RUNOFF
(D.A. = 47.0 SQUARE MILE)
BASED ON PARKER RIVER FLOWS
1946-1985 (D.A. = 21.6 SQUARE MILES)

	Mean		Maximum		Minimum	
	Inches	CFS	Inches	CFS	Inches	CFS
Jan	2.33	(93.9)	6.25	(251.7)	.16	(6.5)
Feb	2.52	(112.6)	5.94	(264.7)	.25	(11.4)
March	4.85	(195.3)	12.19	(490.4)	2.23	(89.8)
April	4.22	(175.3)	8.20	(340.7)	1.31	(54.7)
May	2.69	(108.2)	8.14	(327.7)	1.03	(41.7)
June	1.44	(59.9)	7.20	(299.4)	.22	(9.2)
July	.47	(18.9)	2.10	(84.6)	.05	(2.1)
Aug	.29	(11.7)	.97	(39.0)	.02	(.8)
Sept	.30	(12.6)	3.39	(141.0)	.006	(.3)
Oct	.68	(27.3)	6.64	(266.9)	.009	(.4)
Nov	1.46	(61.0)	4.54	(188.7)	.05	(2.0)
Dec	2.17	(87.4)	6.31	(253.0)	.09	(3.8)
Annual	23.4	(79.8)	41.1	(140.6)	8.4	(18.2)

Evapotranspiration and ground water depletion are greatest during the summer, and runoff the least during the summer and early fall seasons. Ground water recharge takes place during the winter and spring seasons with streamflows normally highest during the spring snowmelt season with nearly 40 percent of average annual runoff occurring during March and April.

b. Peak Discharges. Peak nontidal inflows to the Saugus River tidal estuary are comprised of:

(1) The Saugus River flow where it enters estuary tide water in the vicinity of Woodbury Street crossing.

(2) Inflow from the lower basin, made up of discharge, direct to tide water, from several short tributaries and urban drains, plus rainfall directly on the extensive tidal basin water surface. Plate 1 shows subwatershed delineation and areas.

Peak Saugus River inflow to tide water, with a drainage area of about 26 square miles, is a function of both rainfall intensity and limiting channel conveyance capacity. Peak riverflows are also modified by extensive wetlands in the head waters of the basin in Wakefield and by numerous lakes and ponds throughout the basin. Peak discharge frequencies of the Saugus River, for projected future conditions, were estimated by statistical analysis of discharge records of the neighboring Aberjona River at a USGS gage in Winchester, Massachusetts. Analysis was made using a Log Pearson type III statistical distribution and results were transferred to the Saugus River. The drainage area of the Aberjona River at the gage site is 24.1 square miles, or nearly equal to that of the upper Saugus watershed area of 25.7 square miles. The computed discharge frequencies are listed in table 5.

TABLE 5

PEAK DISCHARGE FREQUENCIES
UPPER SAUGUS RIVER BASIN
(Based on Aberjona River Data)*

Frequency		Discharges
Percent	Years	CFS
50	2	300
10	10	700
5	20	900
2	50	1,300
1	100	1,600
0.5	200	2,000

Note: *Upper Saugus D.A. = 25.7 square miles
Aberjona D.A. = 24.1 square miles

Local inflow to tide water from the remaining 21 square miles of lower basin area is quite indeterminate. Surficial soils in this area are relatively permeable, but with relatively high water table dependent on tide. Also, gravity flow from much of the area is dependent on tide. Presently, when tide is in, considerable storm drainage goes into temporary ground water storage or localized surface ponding followed by drainage when the tide goes out. Some of the low areas are served by "flap-gated" drains to prevent backflow when tide is in. It is projected that with increased development and improved storm drainage systems in the future, peak discharges from the local area will increase significantly. Peak discharge frequencies from the local 21-square mile area, under estimated present, projected future, and potential maximum conditions, were estimated using a combination of: (1) runoff rates equal to average 6-hour rainfall excess frequencies (6-hour rainfall losses) and comparing the results with (2) peak recorded runoff and frequency relations for small drainage areas; the 21-square miles being comprised of numerous small subareas draining individually to the estuary. Reference: "Floodflow Formulas For Urbanized and Unurbanized Areas of Connecticut," by L. A. Weiss, August 1975. The resulting, quite subjective, peak inflow frequencies from the local area used during current studies are listed in table 6.

TABLE 6

PEAK DISCHARGE FREQUENCIES
LOWER SAUGUS RIVER BASIN
(D.A. = 21 SQUARE MILES)

Frequency %	Frequency Yrs	Est. Present Q CFS	Projected Future Q CFS	Est. Max. Potential Q CFS
50	2	550	800	1,100
10	10	1,200	2,000	2,800
5	20	1,500	2,700	3,800
2	50	2,000	3,900	5,500
1	100	2,300	4,700	7,000
0.5	200	2,700	5,400	7,700

5. TIDAL ESTUARY

a. General. The lower 4.7 miles of the Saugus River, and the entire 3 mile length of its Pines River tributary, are tidal estuaries. These estuaries and their adjacent saltwater marshes cover a total area in the lower Saugus River basin of over 1,600 acres. These extensive saltwater wetlands have hydrologic as well as environmental significance.

b. Area-Capacity Curves. The total water surface area in the estuary varies from about 260 acres at normal low tide (EL -4.5 feet NGVD) to about 700 acres at normal high tide (EL +5.0 feet NGVD). Similarly, the approximate water surface area for a spring tide ranges from 230 acres at low tide (EL -5.2 feet NGVD) to 940 acres at high tide (EL +5.8 feet NGVD). Under a storm tide condition, with a level of elevation 8.0 feet NGVD, the water surface area (total flooded area) is about 1,800 acres. Developed area-capacity curves for the Saugus River basin estuary are shown on plate 2. These curves were developed by the Corps from available topographic mapping for the region in association with aerial photographs of the estuary taken under a range of tide level conditions.

During the design phase, detailed mapping and delineations of the tidal estuary with topographic overlays of no less than 2-foot contours will be undertaken to delineate the wetlands; these maps will also be used to further refine the area-capacity curves, if needed, which were developed for this feasibility report.

6. TIDAL HYDROLOGY

a. Astronomical Tides. In the study area (figure 1), tides are semidiurnal, with two high and two low waters occurring during each lunar day (approximately 24-hours 50-minutes). The resulting tide range is constantly varying in response to the relative positions of the earth, moon, and sun; the moon having the primary tide producing effect. Maximum tide ranges occur when the orbital cycles of these bodies are in phase. A complete sequence of tide ranges is approximately repeated over an interval of 19 years, which is known as a tidal epoch.

(1) Boston. At the National Ocean Survey (NOS) tide gage in Boston, Massachusetts (the one nearest to the study area), the mean range of tide and the mean spring range of tide are 9.5 feet and 11.0 feet, respectively. However, the maximum and minimum predicted astronomic tide ranges at Boston have been estimated at about 14.7 and 5.0 feet, respectively, using the Coastal Engineering Research Center (CERC) report, entitled "Tides and Tidal Datums in the United States," SR No. 7, 1981. The frequency of astronomic tidal fall (the difference between consecutive high and low tides) as determined by CERC is presented in figure 2. The variability of astronomical tide ranges is a very significant factor in tidal flooding potentials throughout the area under study. This is explained further in section 6d.

Because of the continual variation in water level due to the tides, several reference planes, called tidal datums, have been defined to serve as a reference zero for measuring elevations of both land and water. Tidal datum information for Boston is presented on figure 3 and table 7. These data were compiled using currently available NOS tidal benchmark data for Boston along with the previously mentioned CERC report. The epoch for which the National Ocean Survey has published tidal datum information for Boston is 1960-78. A phenomenon that has been observed through tide gaging and tidal benchmark measurements is that sea level is apparently rising with respect to the land along most of the U.S. Coast. At the Boston National Ocean Survey tide gage, the rise has been observed to be slightly less than 0.1 foot per decade. Sea level determination is generally revised at intervals of about 25 years to account for the changing sea level phenomenon (rising sea level is further discussed in section 14).

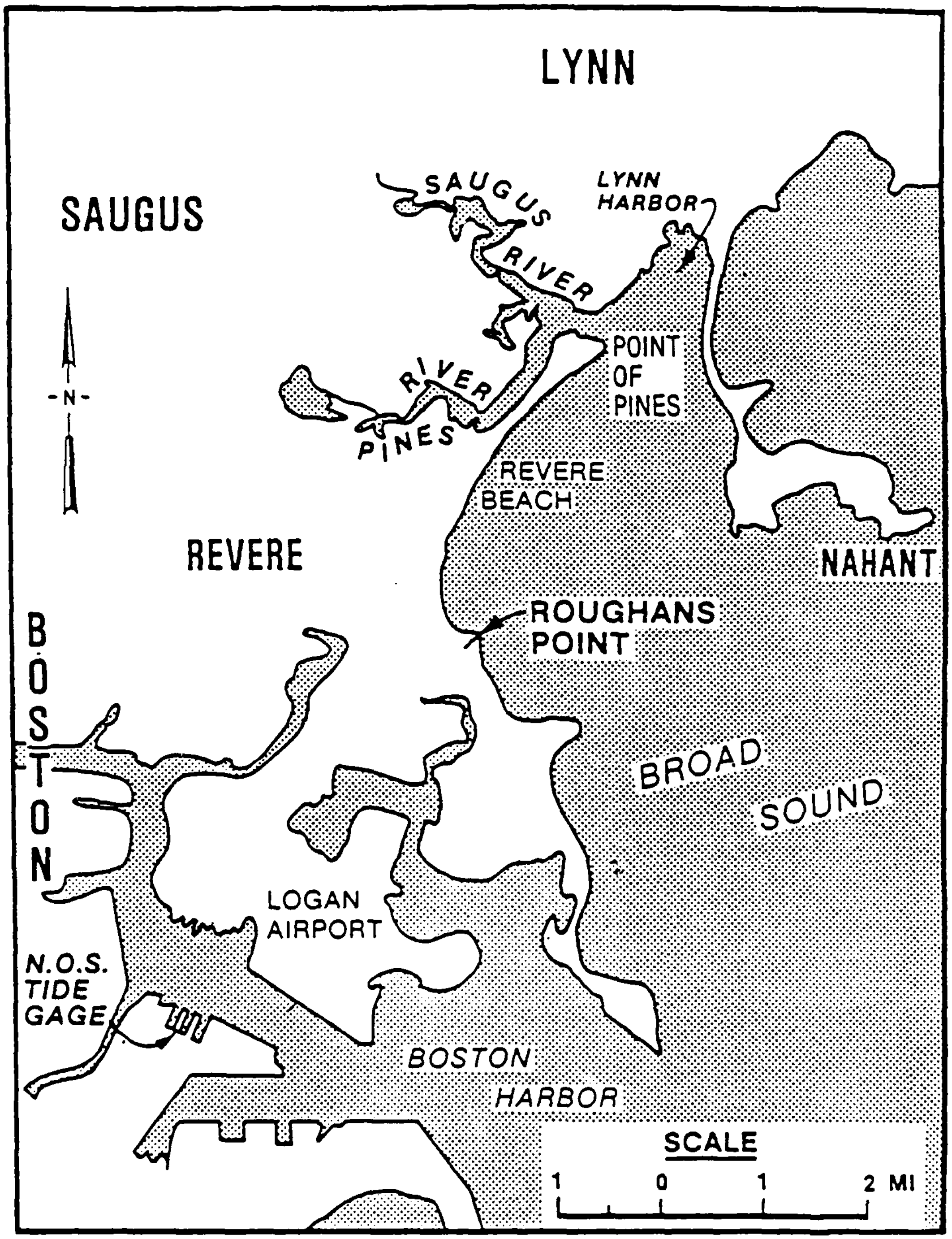


FIGURE 1 Study area vicinity

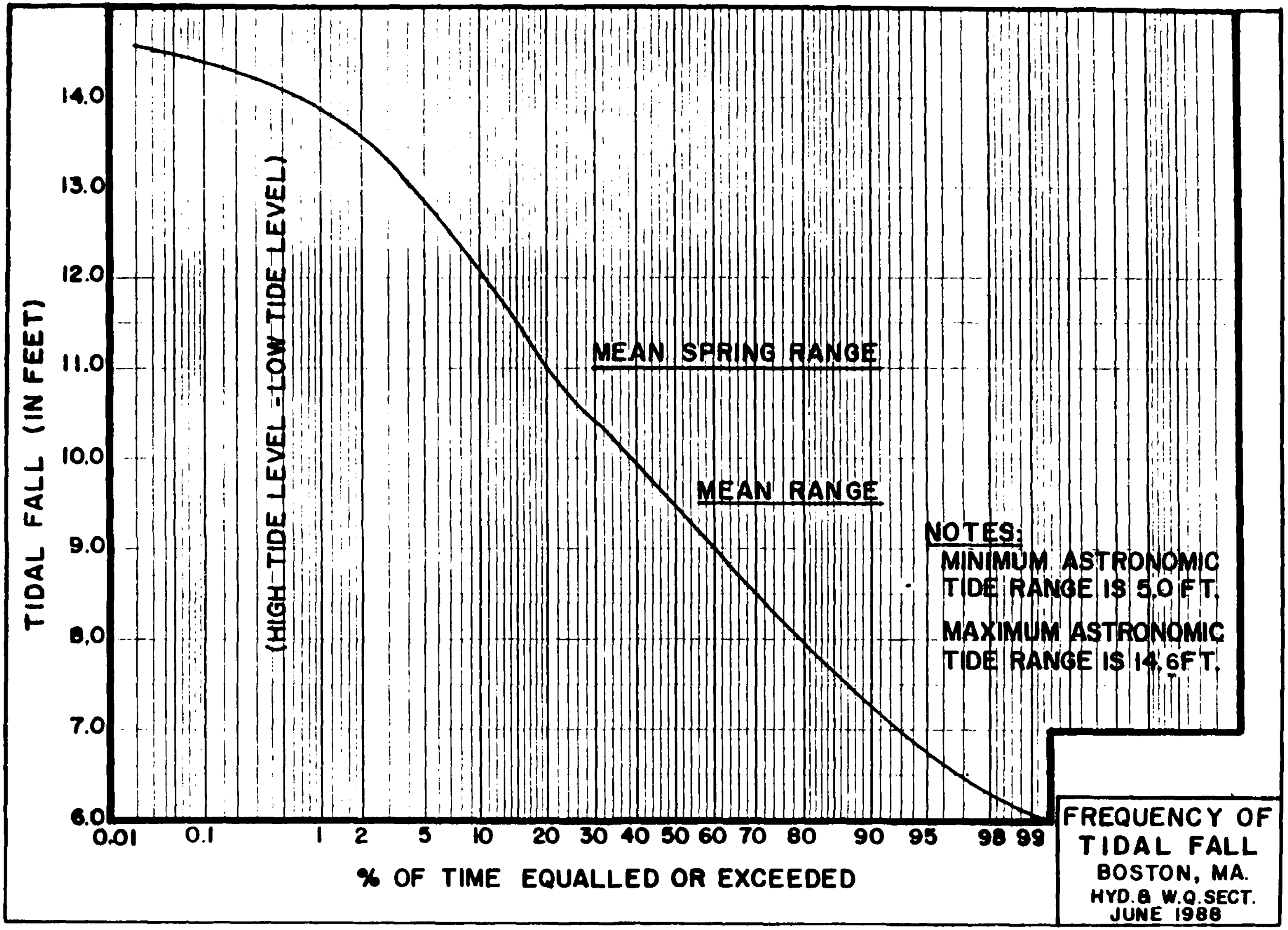
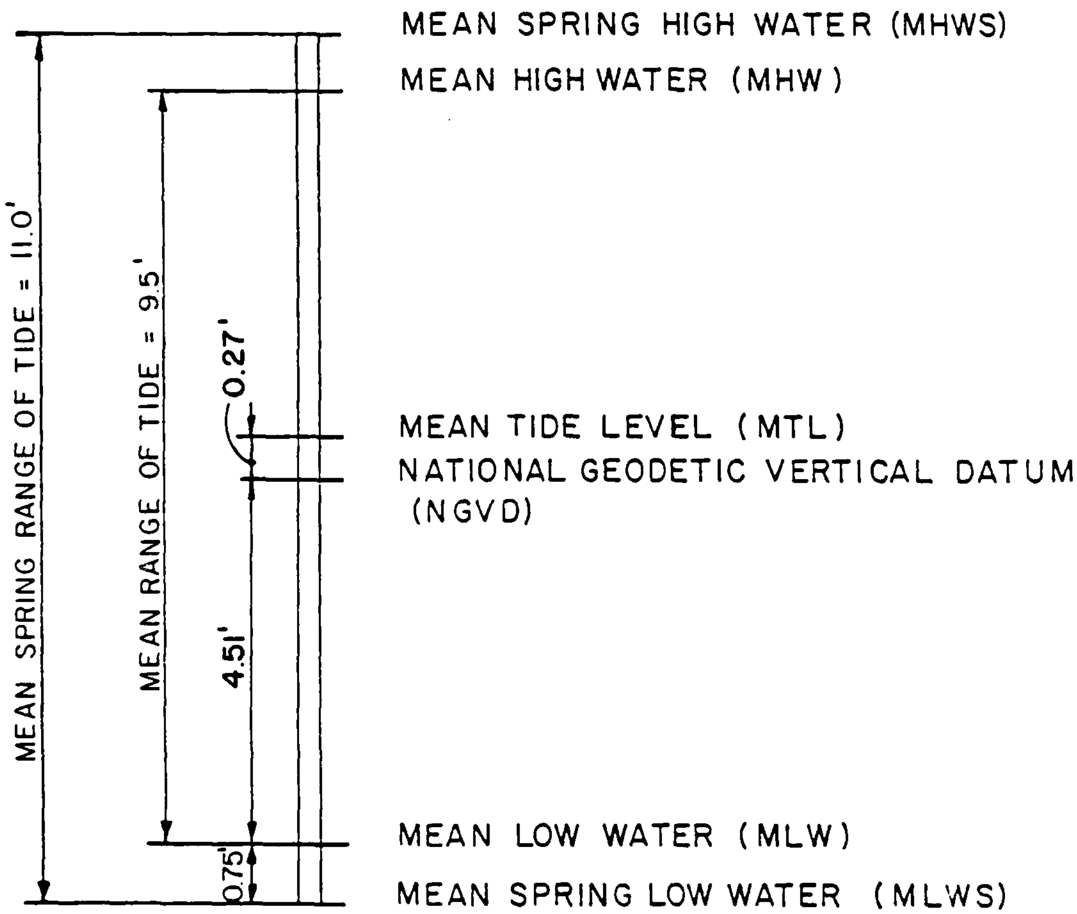

FREQUENCY OF
TIDAL FALL
BOSTON, MA.
HYD.& W.Q.SECT.
JUNE 1988
TIDAL FALL (IN FEET)
(HIGH TIDE LEVEL - LOW TIDE LEVEL)
% OF TIME EQUALLED OR EXCEEDED
MEAN SPRING RANGE
MEAN RANGE
NOTES:
MINIMUM ASTRONOMIC TIDE RANGE IS 5.0 FT.
MAXIMUM ASTRONOMIC TIDE RANGE IS 14.6 FT.
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
0.01
0.1
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
FIGURE 2

FIGURE 3

TIDAL DATUM PLANES
BOSTON, MASSACHUSETTS
NATIONAL OCEAN SURVEY TIDE GAGE

(BASED UPON 1960-78 NOS TIDAL EPOCH)



NEW ENGLAND DIVISION
U.S. ARMY, CORPS OF ENGINEERS
WALTHAM, MASS. MARCH 1985

TABLE 7

BOSTON TIDAL DATUM PLANES
NATIONAL OCEAN SURVEY TIDE GAGE
(BASED UPON 1960-78 NOS TIDAL EPOCH)

	Tide Level (Ft. NGVD)
Maximum Predicted Astronomical High Water	7.5
Mean Spring High Water	5.8
Mean High Water (MHW)	5.0
Minimum Predicted Astronomical High Water	2.7
Mean Tide Level (MTL)	0.3
National Geodetic Vertical Datum (NGVD)	0.0
Maximum Predicted Astronomical Low Water	-2.4
Mean Low Water (MLW)	-4.5
Mean Spring Low Water (MLWS)	-5.2
Minimum Predicted Astronomical Low Water	-7.1

(2) Study Area. During the summer of 1984, NED in cooperation with the U.S. Geological Survey, supervised the placement and operation of five recording tide gages in the study area. Plate 9 shows the location of these gages. Two of the gages, Simpson's Pier and Bay Marine Lobster, were located outside the river system at Roughans Point and in Lynn Harbor, respectively. The other three gages (Fox Hill Drawbridge, Broad Sound Tuna, and Atlantic Lobster) were located in the Saugus-Pines River system. All these gages were in operation from June to October 1984. The Fox Hill Drawbridge and Bay Marine Lobster recording gages have continued in operation through the present time. In addition, staff gages were installed at the Town Line Brook tide gate, Foxhill Drawbridge, and at Lincoln Avenue in August 1985.

The tide range, timing phase and mean tide level of the study area recording gages and the staff gage at Lincoln Avenue are very close to those of the Boston gage. Water levels at high tide are all within several tenths of a foot, and the phases at high tide are all within several minutes. In general, for normal non-storm tide conditions, the smaller the tide range, the less reduction there is in tide heights as one proceeds upstream from the rivers mouth. Mean tide range produces nearly the same heights inland as at the coast. Normal spring high tides at the Town Line Brook gage near the Seaplane Basin appear to be about one-half foot lower than at Boston and the timing appears to be about 50-minutes later than Boston. This seems due to the restrictive channel opening at the abandoned highway embankment and the relatively large storage available in the Saugus marsh. The largest differences occur at low water where the river gages show a distinctly higher and later low tide, relative to Boston. Plots of typical recording gage data are shown in WES Technical Report CERC-86-8, "Frequency of Coastal Flooding at Roughans Point, Broad Sound, Lynn Harbor, and the Saugus-Pines River System." Tables 8 and 9 list representative normal high and low tide data, respectively, as collected throughout the study area and as compared to the Boston NOS gage. Table 10 provides a summary of the tidal heights and timing relationships for normal high tides within the estuary.

TABLE 8

MAXIMUM NORMAL TIDE ELEVATION* AND TIMING** DATA FOR SAUGUS RIVER ESTUARY

Date / Location	7-29-84 Elev.	Time	8-16-84 Elev.	Time
Boston	6.67	23.8	4.81	2.0
Fox Hill Drawbridge	6.7	23.9	4.7	2.0
Broad Sound Tuna	6.7	23.9	5.0	2.0
Atlantic Lobster	6.5	24.2	4.9	2.1

Date / Location	9-17-85 Elev.	Time	10-15-85 Elev.	Time
Boston	6.19	12.9	7.45	11.5
Fox Hill Drawbridge	6.55	13.0	7.67	11.6
Lincoln Avenue Bridge	6.1	13.3	7.37	11.6
Town Line Tide Gate	6.1	13.3	7.08	12.3

Date / Location	11-13-85 Elev.	Time	12-12-85 Elev.	Time
Boston	7.2	11.2	7.96	10.7
Fox Hill Drawbridge	7.1	11.2	8.13	10.9
Lincoln Avenue Bridge	6.89	11.2	7.89	10.9
Town Line Tide Gate	6.64	11.6	--	--
East Saugus	--	--	6.0 (est)	--

Date / Location	11-3-86 Elev.	Time	11-14-86 Elev.	Time
Boston	6.67	11.3	4.48	9.6
Fox Hill Drawbridge	6.88	11.3	4.69	9.5
Lincoln Avenue Bridge	6.67	11.4	4.55	9.5
Town Line Tide Gate	6.46	11.7	4.70	9.6
East Saugus	--	--	--	-

Date / Location	12-2-84 Elev.	Time	4-16-87 Elev.	Time
Boston	--	--	--	--
Fox Hill Drawbridge	6.78	10.9	5.6	13.7
Lincoln Avenue Bridge	6.57	10.9	5.57	14.0
Town Line Tide Gate	6.36	11.4	5.5	14.5

Notes: *All elevations are in feet NGVD.
**Timing is in hours and tenths, Eastern Standard Time, 24-hour clock.

TABLE 9

MINIMUM NORMAL TIDE ELEVATION* AND TIMING** DATA
FOR SAUGUS RIVER ESTUARY

Date Location	7-30-84 Elev.	 Time	8-16-84 Elev.	 Time
Boston	-6.3	6.4	-3.8	8.1
Fox Hill Drawbridge	-6.3	6.3	-3.7	7.9
Broad Sound Tuna	-5.7	6.8	-3.6	8.1
Atlantic Lobster	-5.2	7.3	-3.6	8.4
Date **Location**	**5-7-87** **Elev.**	 **Time**	**5-18-87** **Elev.**	 **Time**
Boston				
Fox Hill Drawbridge	-3.0	13.0	-5.1	9.8
Lincoln Avenue Bridge	-3.0	13.3	--	--
Town Line Tide Gate	-2.8	13.5	-3.1	10.2

Notes: *All elevations are in feet NGVD.
**Timing is in hours and tenths, Eastern Standard Time, 24-hour clock.

TABLE 10

NORMAL HIGH TIDE COMPARISON

Location	Tide Difference From Boston (feet)	Time Difference From Boston (minutes)
Boston	0	0
Fox Hill Drawbridge	0 to -0.1	0 to 15
Lincoln Avenue Bridge	-0.1 to -0.3	5 to 15
Broad Sound Tuna	0 to 0.3	0 to 5
Atlantic Lobster	0 to 0.2	5 to 15
Town Line Brook	-0.4 to -0.6	30 to 50
East Saugus	-1.5 to -2.5	50 to 80

Tide gage measurements during non-storm periods have verified that there is little difference between ocean tide levels and interior estuary tide levels. Therefore, for equivalent tide level changes, there is a required saltwater interchange between ocean and estuary. Storage capacity in the lower Saugus River Basin estuary between normal low tide (EL -4.5 feet NGVD) and normal high tide (EL 5.0 feet NGVD) is about 4,600 acre-feet. Therefore, with a normal tide cycle of about 12 hours there is an average rate of saltwater interchange of about 9,400 cfs. Under midtide rate of change of 2.4 feet per hour, the required interchange rate is about 15,000 cfs. Comparatively, under a spring tide range of 11 feet, total storage interchange is about 5,500 acre-feet in 6-hours for an average interchange rate of 11,000 cfs, and under a midtide rate of change of 2.7 feet per hour, the required interchange rate is about 17,000 cfs.

Placing the hydrology and tidal hydraulics of the estuary in perspective, the average rate of saltwater interchange to the estuary is over 100 times the average freshwater inflow from the basin (9,400 cfs vs 80 cfs). Even under the rare condition of a one percent chance Saugus River flow, coincident with a one percent chance local runoff, the peak freshwater inflow to the estuary would be about 43 percent (4,000 vs 9,400 cfs) of the average rate of saltwater tidal interchange. Tidal hydrology of the Saugus River estuary and its resulting environment are more a function of the hydraulics of tide water interchange than of basin runoff. Similarly, the hydraulic requirements of any structural improvements for flood control are more a function of the requirements for tidal interchange than for freshwater discharge.

b. Storm Types. Two distinct types of storms, distinguished primarily by their place of origin as being extratropical and tropical cyclones, influence coastal process in New England. These storms can produce above normal water levels and waves and must be recognized in studying New England coastal problems.

(1) Extratropical Cyclones. These are the most frequently occurring variety of cyclones in New England. Low pressure centers frequently form or intensify along the boundary between a cold dry continental air mass and a warm moist marine air mass just off the coast of Georgia or the Carolinas and move northeastward more or less parallel to the coast. These storms derive their energy from the temperature contrast between cold and warm air masses. The organized circulation pattern associated with this type of storm may extend for 1,000 to 1,500 miles from the storm center. The

wind field in an extratropical cyclone is generally asymmetric with the highest winds in the northeastern quadrant. When the storm center passes parallel and to the southeast of the New England coastline and the highest onshore windspeeds are from the northeast, these storms are called "northeasters" or "nor'easters" by New Englanders. As the storm approaches and passes, local wind directions may vary from southeast to slightly west of north. Coastlines exposed to these winds can experience high waves and extreme storm surge. Such storms are the principal tidal flood producing events throughout the study area. Other storms which take a more inland track can have high winds from the southeast. These are referred to as "southeasters." In the area under study, these storms do not generally produce as much storm surge and wave action as "northeasters" due to more limited fetch. The prime season for severe extratropical storms in New England is November through April.

(2) Tropical Cyclones. These storms form in a warm moist air mass over the Caribbean and waters adjacent to the West Coast of Africa. The air mass is nearly uniform in all directions from the storm center. Energy for the storm is provided by the latent heat of condensation. When the maximum windspeed in a tropical cyclone exceeds 75 mph, it is labeled a hurricane. Wind velocity at any position can be estimated based upon the distance from the storm center and the forward speed of the storm. The organized wind field may not extend more than 300 to 500 miles from the storm center. Recent hurricanes affecting New England generally have crossed Long Island Sound and proceeded landward in a generally northerly direction. However, hurricane tracks can be erratic. The storms lose much of their strength after landfall. For this reason the southern coast of New England experiences the greatest surge and wave action from the strong southerly to easterly flowing hurricane winds. However, on very rare occasions, reaches of coastline in eastern and northern New England may experience some storm surge and wave action from the weakened storm. Hurricanes have not been a principal cause of tidal flooding in the greater Boston area. The hurricane and tropical storm season in New England generally extends from August through October.

c. Winds. An estimate of windspeed is one of the essential ingredients in predicting wave heights. The most accurate estimate of winds at sea, which generate waves and propel them landward, is obtained by utilizing isobars of barometric pressure recorded during a given storm. However, actual recorded windspeed and direction data observed at a land based coastal meteorological station can serve as a useful guide when more locally generated waves and currents are of interest. The disadvantage with using land based wind records is that they may not be totally indicative of wind velocities at the sea-air interface where the waves are generated. However, often they are the only available source of information and adjustments must be made to develop over water estimates from the land based records. Also, when estimating wave overtopping of coastal structures, it is necessary to utilize local wind conditions. These local winds help determine how much of the runup from breaking waves is blown over the structure.

(1) Percent Occurrence of Wind Direction and Speed. The National Weather Service (NWS) has recorded 31 years of hourly one-minute average windspeed and direction data at Logan International Airport in Boston, Massachusetts from 1945 through 1979. Logan Airport, which is adjacent to the study area, is the closest location to the project for which relatively complete, systematically recorded, wind data are available. The windspeed data were adjusted to a standard 33-foot observation height and one-minute average windspeeds were converted to one-hour average windspeeds. Since Logan International Airport is almost directly adjacent to the ocean, no land to sea conversion was applied. However, a wind stability correction was made for all fetches of interest. All adjustments were made in accordance with ETL 110-2-305 on the subject of determining wave characteristics on sheltered waters. Utilizing these one-hour average wind data, the percent occurrence of wind direction and windspeed range were computed. Since only onshore winds at the project are of interest, the wind directions utilized in this analysis were limited to those between northeast (NE) and southeast (SE). This analysis, results of which are shown in table 11 and figure 4, indicated that the principal onshore wind direction for windspeeds less than or equal to 5 mph is from the SE and for windspeeds greater than 5 and less than or equal to 15 mph, it is from the ESE. Winds greater than 15 and less than or equal to 20 mph generally come from the E. Winds greater than 20 mph come from the NE. The maximum average windspeed (11.8 mph) is from the NE and the greatest maximum speed was 68.7 mph from the SE. Overall average speed is 10.5 mph. Table 11 also shows the resultant wind direction for various windspeed ranges. The resultant wind direction is a vector quantity computed using the product of windspeed and direction. It is an indicator of net air movement past a given location. Overall, the resultant wind direction is from the E. However, winds greater than 20 mph have a more ENE resultant. The greatest percentage of windspeeds is shown to be greater than 10 and less than or equal to 15 mph.

TABLE 11

ADJUSTED HOURLY WIND OBSERVATIONS BETWEEN NE AND SE AT BOSTON, MASSACHUSETTS
(One-Hour Average Values)

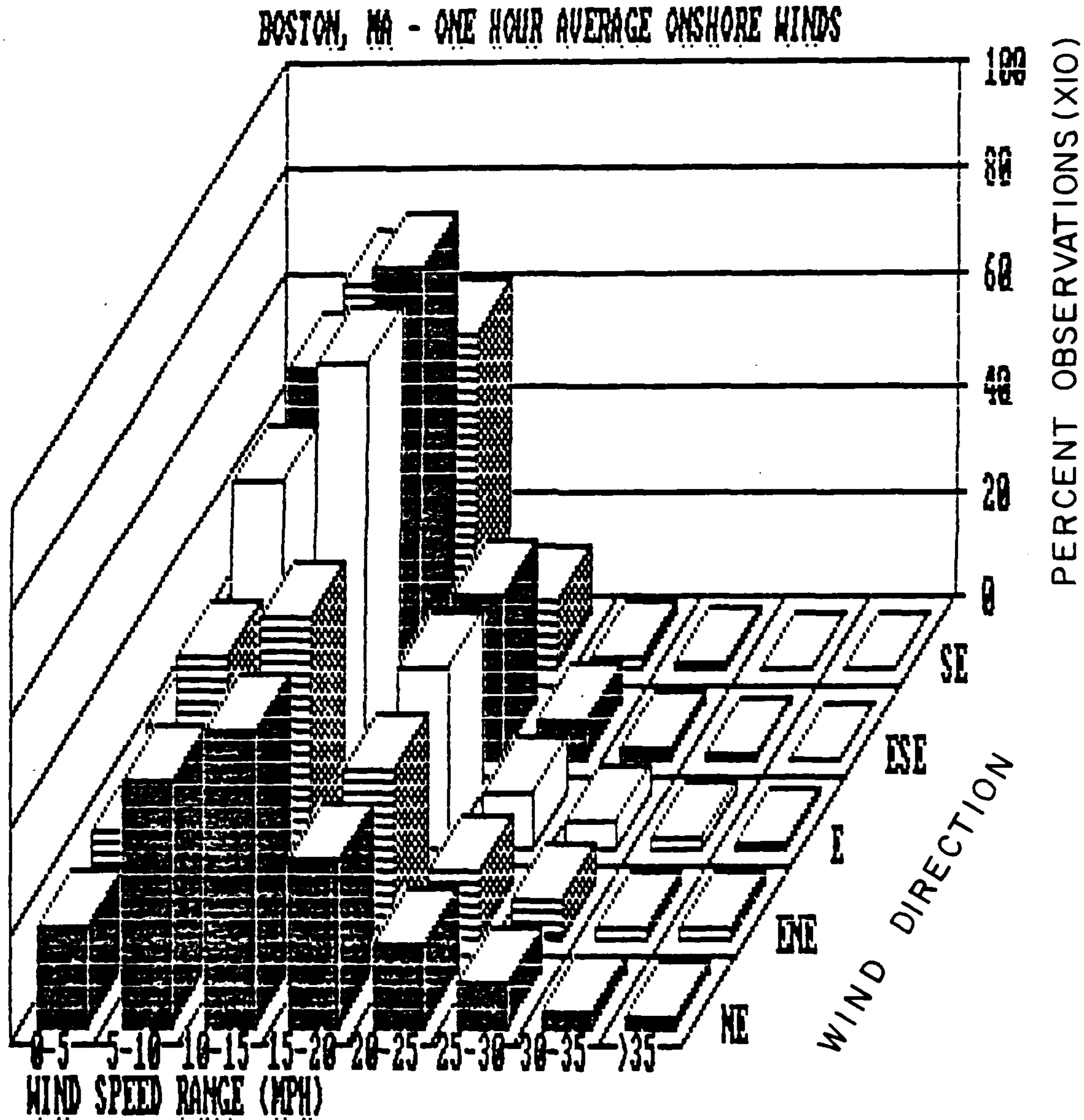
PERCENT OF ONSHORE WINDSPEED AND DIRECTION OBSERVATIONS (X 10)

Direction	Windspeed Range (Miles Per Hour) 0-5	5-10	10-15	15-20	20-25	25-30	30-35	Over 35	All Inclusive	Avg Speed (mph)	Max Speed (mph)
NE	19	46	55	31	16	8	3	2	179	11.8	54.3
ENE	20	52	59	31	13	7	2	2	185	11.3	49.2
E	23	69	91	33	10	5	2	1	234	10.7	55.3
ESE	22	73	92	30	7	2	1	0	227	10.0	49.2
SE	24	72	63	13	2	1	0	0	174	8.7	68.7
NE-SE	108	313	360	136	48	22	7	5	1,000	10.5	68.7
Resultant Direction:	E	E	E	E	ENE	ENE	ENE	ENE	E		

NOTES: 1. Windspeed ranges include values greater than the lower limit and less than or equal to the higher limit.

2. Onshore winds occur 21 percent of the time; therefore, average annual number of occurrence (A) = percent occurrence times 18.654 (for example: a windspeed range of 0-5 mph from the ENE, A = 2.0 (18.654) = 37).

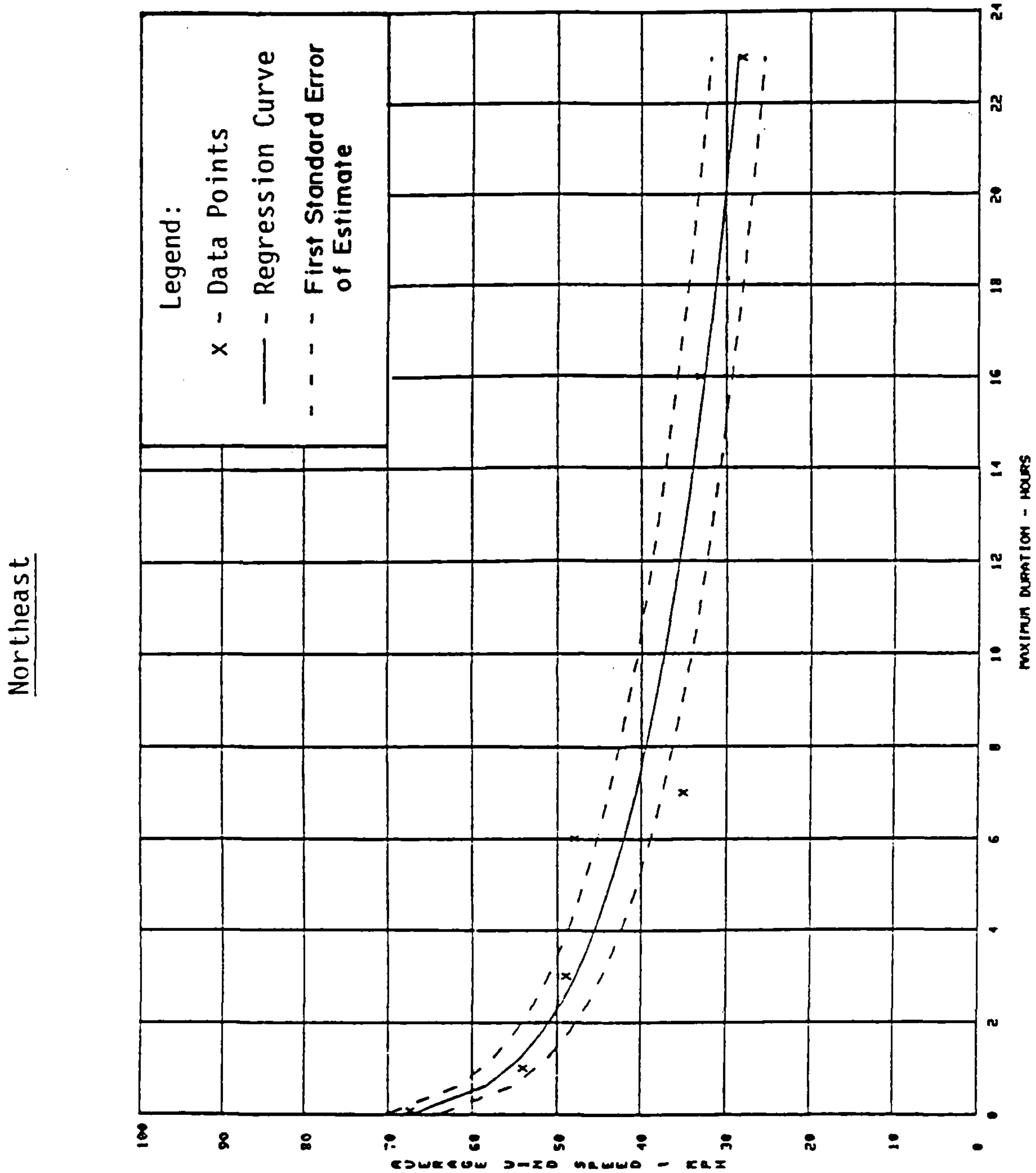
FIGURE 4


BOSTON, MA - ONE HOUR AVERAGE ONSHORE WINDS
100
80
60
40
20
0
PERCENT OBSERVATIONS (XIO)
SE
ESE
E
ENE
NE
WIND DIRECTION
0-5
5-10
10-15
15-20
20-25
25-30
30-35
>35
WIND SPEED RANGE (MPH)

(2) Windspeed Persistence. Additionally, actual windspeed persistence was determined on a directional basis. The resulting maximum windspeed persistence data, shown on figures 5(a) through 5(e), for directions northeast through southeast, indicate the maximum number of consecutive hourly windspeed observations that occurred at a given average speed from a particular direction. This analysis demonstrated that high onshore wind can occur for extended periods of time in the study area. High speed, long duration winds are usually associated with northeasters and, therefore, come from the northeastern quadrant. High intensity, short duration winds have come from the southeast due to hurricane events.

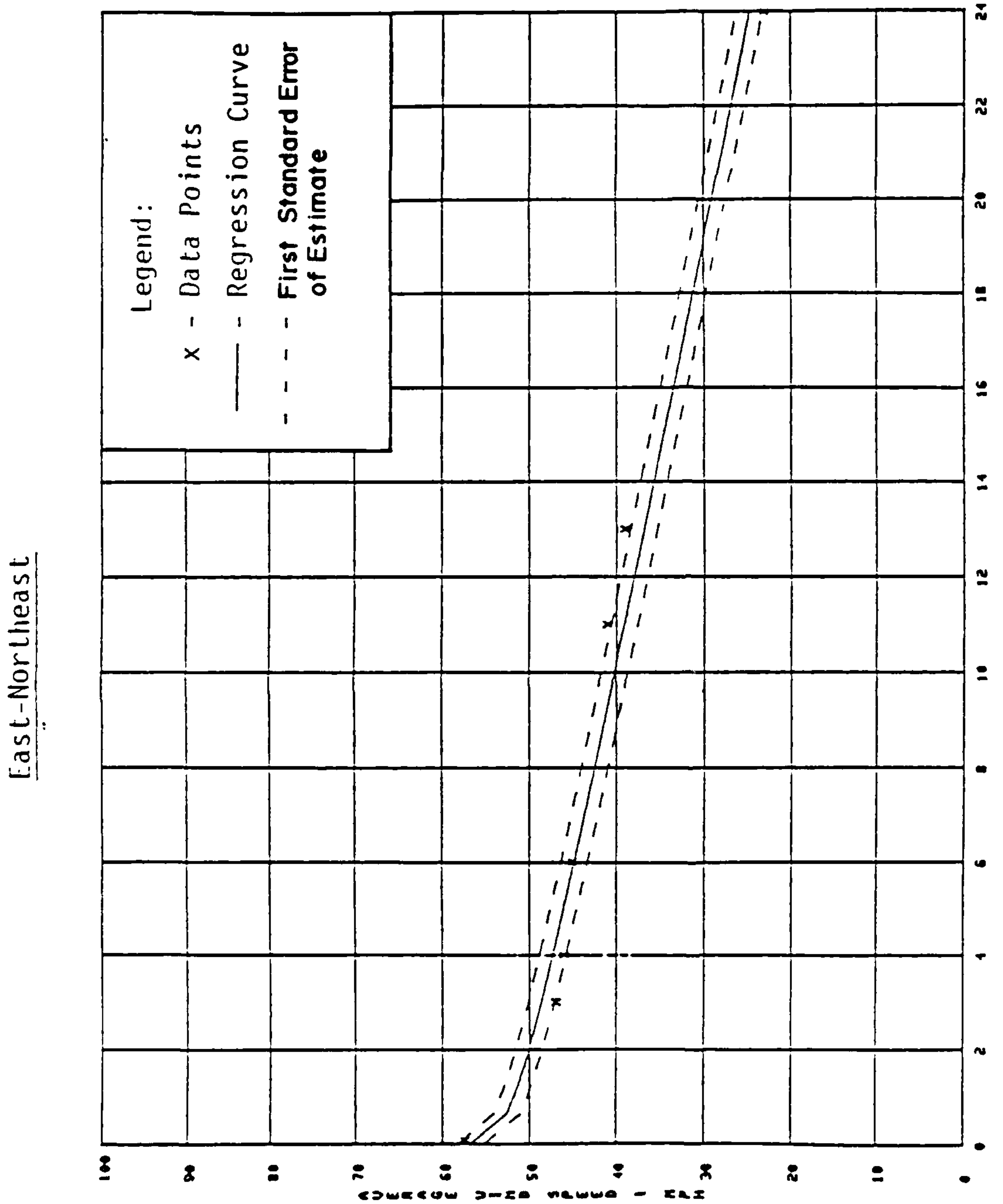
(3) Winds During Historic Storms. When studying overtopping of coastal structures it is useful to examine wind conditions occurring during past flood events in order to get an appreciation for the possible severity of experienced wave overtopping conditions. Table 12 presents National Weather Service (NWS) wind observations recorded at Logan Airport in Boston during notable tidal floods. From these data it can be seen that the strongest winds recorded during flood events generally originated from directions between northeast and east. The greatest fastest-mile (approximately equal to one-minute average speed) listed, 61 mph from the northeast, was recorded on 6 February 1978 during the great "Blizzard of '78." By comparing table 12 with table 15, it can be seen that the stillwater tide levels recorded during these storm events ranged between 10.3 and 8.3 feet, respectively. However, extremely severe onshore winds have occurred during storm events which produced significantly lower observed maximum stillwater tide levels in the study area.

Since the astronomical tide range at the project is so variable, as explained in section 6a, many severe coastal storms occur during periods of relatively low astronomic tides. Thus, even though a storm may produce exceptionally high onshore winds, waves and a tidal surge, the resulting tide level may be less than that occurring during a time of high astronomic tide and little meteorological influence. Table 13 presents wind data recorded at Logan Airport during storms which produced annual maximum surge values of 3 feet or more. For comparison, table 14 lists maximum annual storm surges determined by the NWS in their "Tide Climatology For Boston, MA," November 1982, and associated observed tide levels. It can be seen that recurrence intervals of the maximum observed tide levels recorded on days of maximum annual storm surge were generally less than one year, with only a few storms producing significant tidal flood levels.



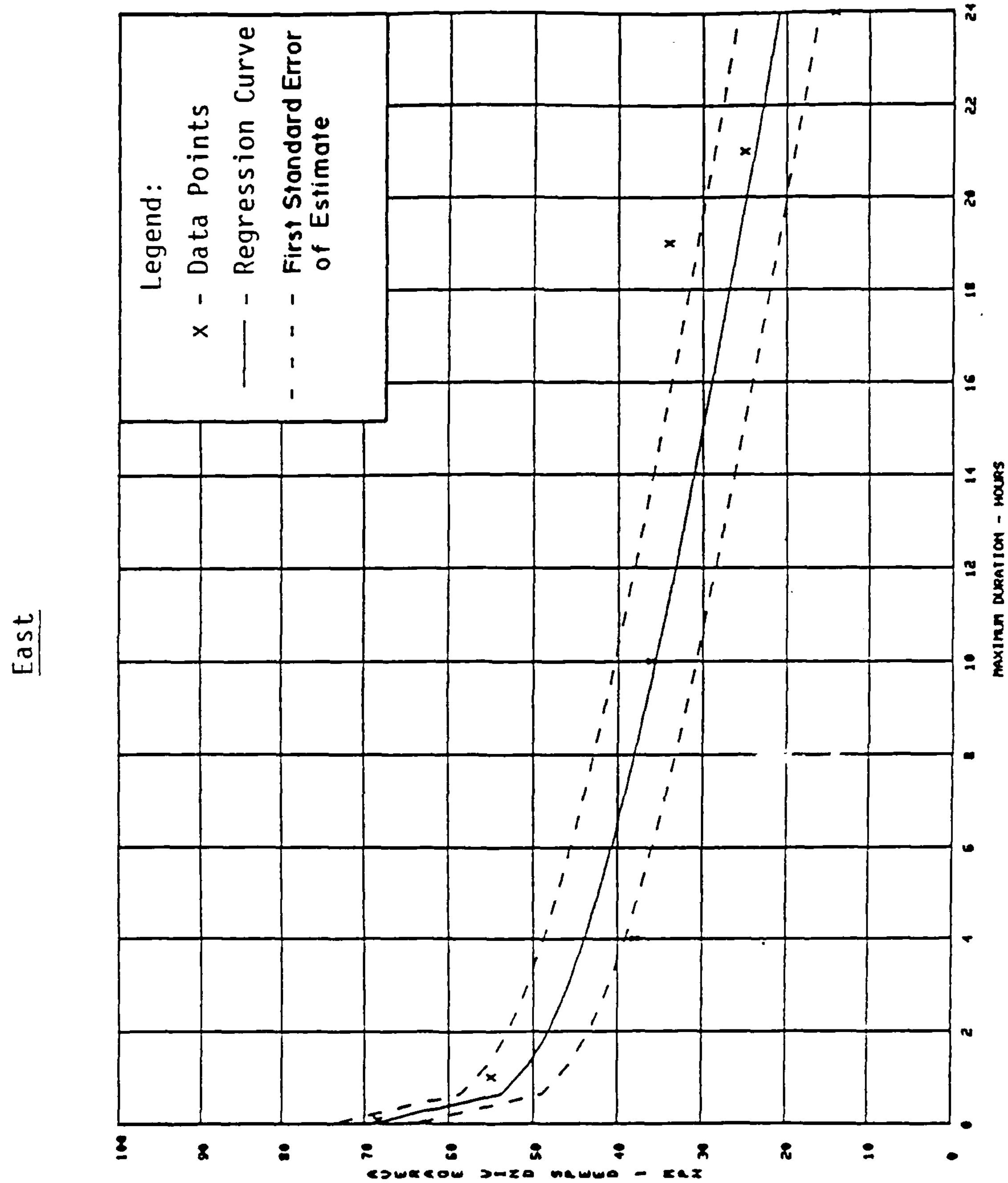
WIND PERSISTENCE
MEASURED AT BOSTON MA.
HYD. & W.Q. SECT. JUNE 1988

FIGURE 5A



WIND PERSISTENCE
MEASURED AT BOSTON, MA.
HYD. & W.Q. SECT. JUNE 1988

FIGURE 5B

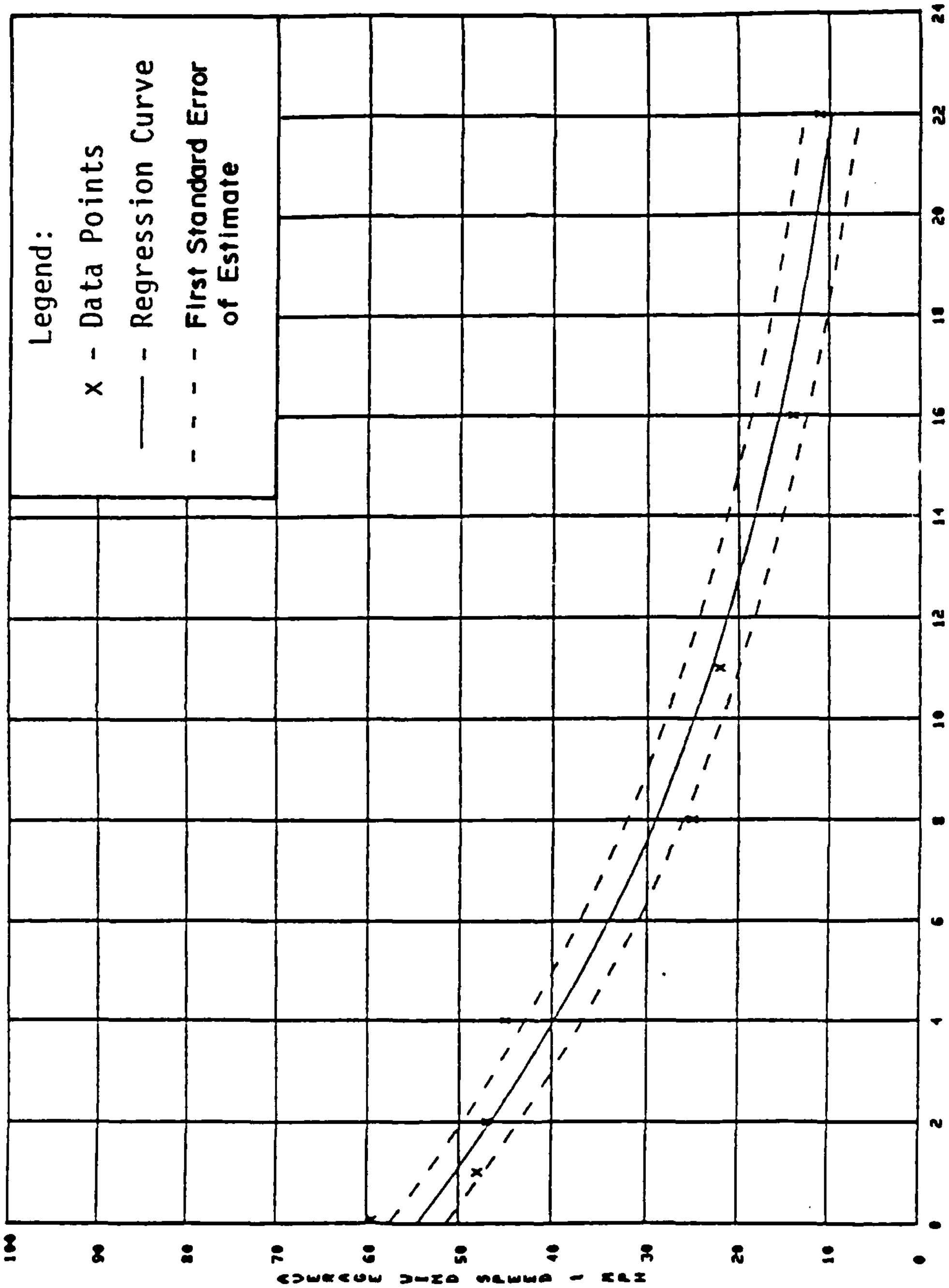


WIND PERSISTENCE
MEASURED AT BOSTON, MA.

HYD. & W.Q. SECT. JUNE 1988

FIGURE 5C

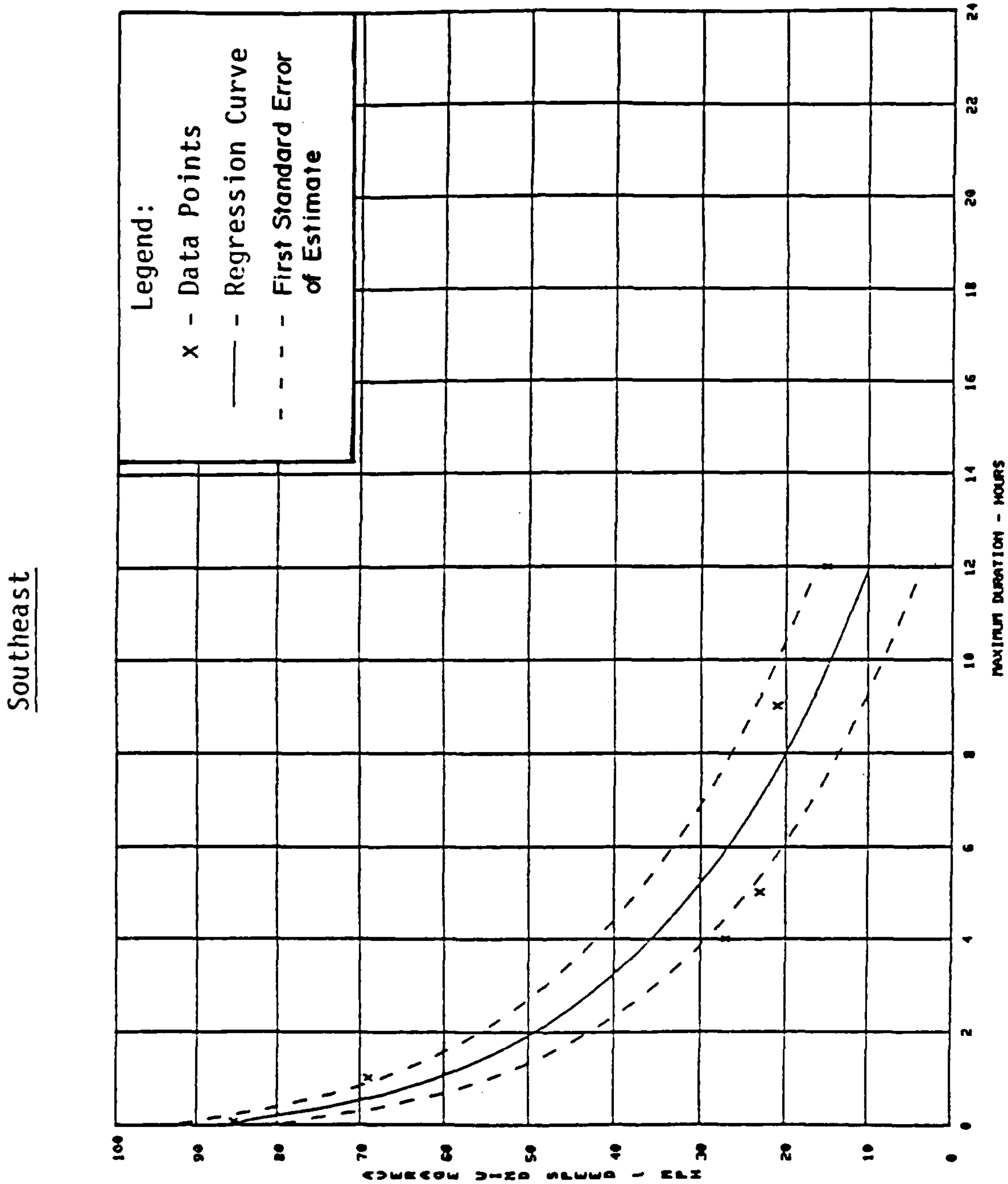
East-Southeast



WIND PERSISTENCE
MEASURED AT BOSTON, MA.

HYD. & W.Q. SECT. JUNE 1988

FIGURE 5D



WIND PERSISTENCE
MEASURED AT BOSTON, MA.
HYD. & W.Q. SECT. JUNE 1988

FIGURE 5E

TABLE 12

BOSTON - LOGAN INTERNATIONAL AIRPORT
NATIONAL WEATHER SERVICE
WIND OBSERVATIONS RECORDED
DURING NOTABLE TIDAL FLOODS

Date	Resultant Direction	Resultant Speed (mph)	Average Speed (mph)	Fastest-Mile Speed (mph)	Fastest-Mile Direction
6 Feb 1978	ENE	28.4	29.3	61	NE
2 Jan 1987	N	15.5	21.8	35	NE
25 Jan 1979	ENE	23.2	24.2	45	E
29 Dec 1959	NE*	--	20.7	34	E
19 Feb 1972	NE	21.1	24.2	47	NE
25 May 1967	NE	34.3	34.7	50	NE
21 Apr 1940	-	--	13.3	43**	NE
20 Jan 1961	NNW*	--	26.7	41	NNE
30 Nov 1944	-	--	13.4	48**	NE
9 Jan 1978	SSW	22.8	28.8	43	SW
16 Mar 1976	ENE	15.4	20.4	35	NE
16 Mar 1956	ENE*	--	28.1	54	NE
6 Apr 1958	WSW*	--	13.8	32	SSE
26 Feb 1979	NE	19.1	19.6	30	NE
2 Dec 1974	ENE	15.7	20.7	38	E
7 Mar 1962	NE*	--	31.6	42	ENE
4 Apr 1973	E	13.0	13.5	31	E
22 Dec 1972	N	13.3	13.5	21	N

* Resultant speed and direction not available for the period prior to 1964; direction shown is prevailing wind direction.

**Fastest-mile not available; value shown is five-minute average speed.

NOTE: Listing is in order of decreasing observed stillwater tide level to provide uniformity with table 15.

TABLE 13

BOSTON LOGAN INTERNATIONAL AIRPORT
NATIONAL WEATHER SERVICE
WIND OBSERVATIONS RECORDED
DURING ANNUAL MAXIMUM SURGE
PRODUCING STORMS
(1922-1979)

Date	Fastest-Mile Speed (mph)	Fastest-Mile Direction	Average Speed (mph)	Prevailing Direction
29 Nov 1945	63*	NE	40.5	-
13 Apr 1961	42	ENE	25.0	NE
6 Feb 1978	61	NE	29.3	ENE
14 Feb 1940	51*	NE	12.7	-
17 Nov 1935	54*	NE	14.9	-
19 Feb 1972	47	NE	24.2	NE
3 Mar 1947	50*	E	13.4	-
4 Mar 1960	45	NE	28.0	N
30 Jan 1966	43	S	22.3	SSE
12 Nov 1968	54	NE	23.9	E
25 Jan 1979	45	E	24.2	ENE
22 Mar 1977	60	NE	19.3	E
25 Nov 1950	74	E	42.4	E
31 Aug 1954	86	SE	31.8	ENE
16 Feb 1958	45	E	28.0	E
15 Nov 1962	37	NW	28.5	NW
16 Mar 1956	54	NE	28.1	ENE
27 Dec 1969	26	E	17.3	WNW
11 Mar 1924	--	-	--	-
30 Jan 1939	43*	NE	12.7	-
17 Feb 1952	50	NE	29.8	NE
7 Mar 1923	--	-	--	-
20 Feb 1927	--	-	--	-
19 Jan 1936	40*	NE	12.6	-
7 Nov 1953	67	NE	30.5	NE
14 Aug 1971	18	E	9.6	E
28 Jan 1973	23	NE	19.4	NE
12 Mar 1959	42	ESE	23.9	SE
16 Apr 1929	--	-	--	-
8 Mar 1931	--	-	--	-

* Fastest-mile not available; value shown is five-minute average speed.

NOTE: Listing in order of decreasing annual maximum storm surge to allow comparison with table 14.

TABLE 14

ANNUAL MAXIMUM STORM SURGE
BOSTON, MASSACHUSETTS
(1922-1979)

Date	Annual Maximum Storm Surge (feet)	Maximum Observed Tide Level for the Day (feet, NGVD)	Recurrence* Interval (years)
30 Nov 1945	4.9	7.6	LT 1
13 Apr 1961	4.7	8.0	1
6 Feb 1978	4.6	10.0	50
14 Feb 1940	4.2	5.0	LT 1
17 Nov 1935	4.1	6.5	LT 1
19 Feb 1972	4.0	9.1	10
3 Mar 1947	3.8	7.2	LT 1
4 Mar 1960	3.8	8.1	2
30 Jan 1966	3.8	5.5	LT 1
12 Nov 1968	3.7	7.7	LT 1
25 Jan 1979	3.7	9.2	13
22 Mar 1977	3.6	5.3	LT 1
25 Nov 1950	3.6	6.4	LT 1
31 Aug 1954	3.5	8.2	2
16 Feb 1958	3.5	7.9	1
15 Nov 1962	3.5	7.9	1
16 Mar 1956	3.4	5.6	LT 1
27 Dec 1969	3.3	6.7	LT 1
11 Mar 1924	3.2	6.2	LT 1
31 Jan 1939	3.2	6.9	LT 1
18 Feb 1952	3.2	7.9	1
7 Mar 1923	3.1	6.9	LT 1
20 Feb 1927	3.1	6.9	LT 1
19 Jan 1936	3.1	5.9	LT 1
7 Nov 1953	3.0	7.4	LT 1
14 Aug 1971	3.0	5.4	LT 1
29 Jan 1973	3.0	6.1	LT 1
12 Mar 1959	2.9	6.5	LT 1
16 Apr 1929	2.8	6.6	LT 1
8 Mar 1931	2.8	6.5	LT 1

* Recurrence interval of observed tide elevations. Obtained from tide stage-frequency relationship, figure 8.

NOTE: LT = Less Than

TABLE 15

MAXIMUM STILLWATER TIDE HEIGHTS
BOSTON, MASSACHUSETTS

Date	Observed Elevation (feet, NGVD)	Adjusted Elevation* (feet, NGVD)	Recurrence*** Interval (years)
7 Feb 1978	10.3	10.4	91
16 Apr 1851	10.1	10.4	63
26 Dec 1909	9.9	10.5	42
2 Jan 1987	9.4	9.4	17
25 Jan 1979	9.3	9.4	14
29 Dec 1959	9.3	9.5	14
27 Dec 1839	9.2**	--	13
15 Dec 1839	9.2**	--	13
19 Feb 1972	9.1	9.2	11
24 Feb 1723	9.1**	--	11
26 Mar 1830	9.0**	--	9
26 May 1967	8.9	9.0	7
21 Apr 1940	8.9	9.2	7
29 Dec 1853	8.9	9.2	7
4 Dec 1786	8.9**	--	7
20 Jan 1961	8.8	9.0	6
30 Nov 1944	8.8	9.1	6
4 Mar 1931	8.8	9.2	6
3 Dec 1854	8.8	9.1	6
3 Nov 1861	8.7	9.1	5
9 Jan 1978	8.6	8.7	4
16 Mar 1976	8.6	8.7	4
17 Mar 1956	8.6	8.8	4
7 Apr 1958	8.5	8.7	4
15 Nov 1871	8.5	9.0	4
23 Nov 1858	8.5	8.9	4
26 Feb 1979	8.4	8.5	3
2 Dec 1974	8.4	8.5	3
7 Mar 1962	8.4	8.6	3
4 Apr 1973	8.3	8.4	2
22 Dec 1972	8.3	8.4	2
28 Jan 1933	8.3	8.7	2

*Observed values after adjustment for changing mean sea level; adjustment made to 1987 mean sea level.

**Approximate value based upon historical account. Record not sufficient to document change of sea level for this time.

***Recurrence interval of observed tide elevations. Obtained from tide stage-frequency relationship, figure 8.

NOTE: Events occurring within about 30 days of a greater tide producing event are excluded from this list. Events recorded during years for which only partial records are available were also excluded.

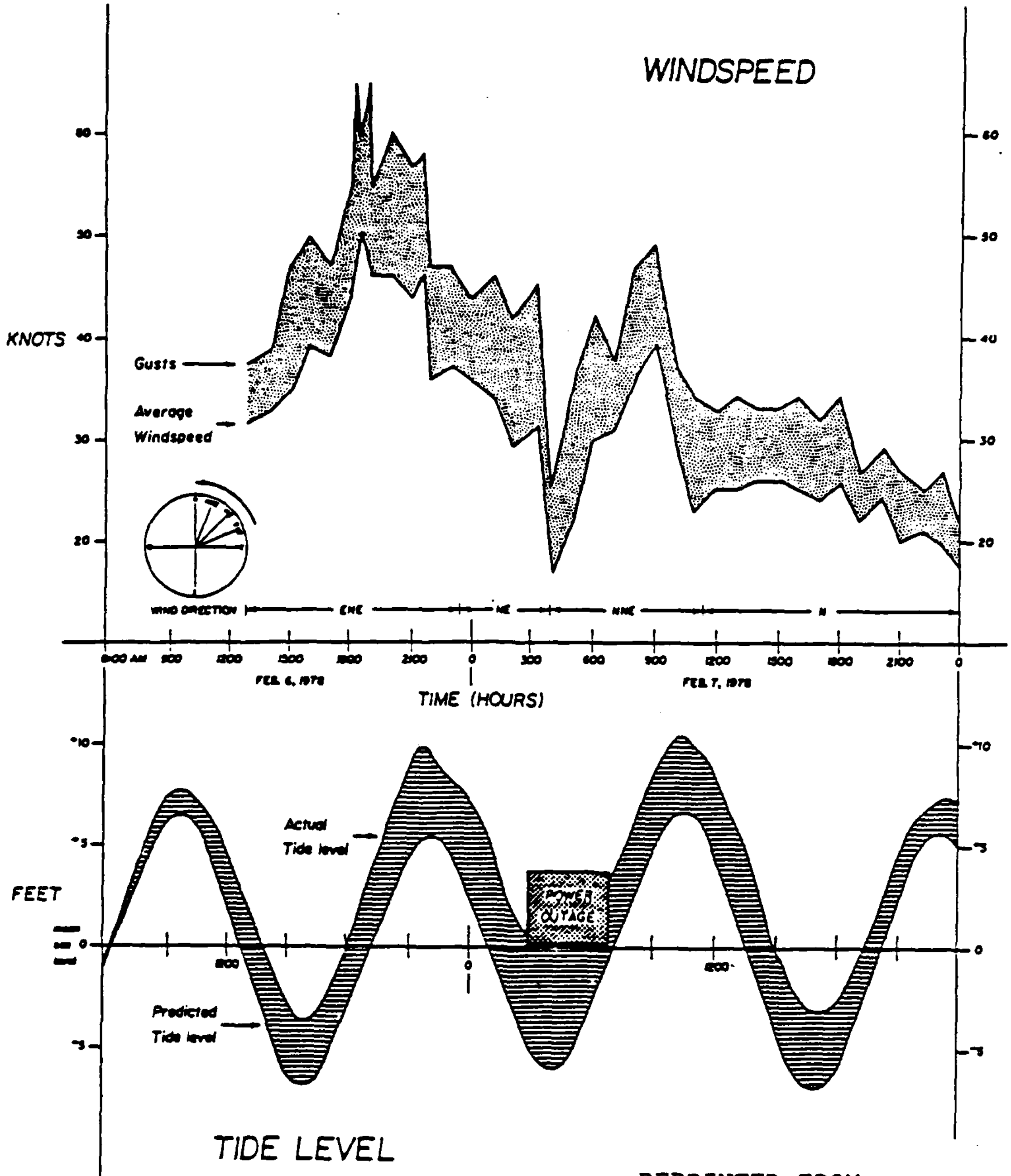
Some of the most severe onshore winds, waves and storm surges are shown to have produced minor tidal flooding, owing to their coincidence with low astronomic tides. A good example of this is the 30 November 1945 event which produced the maximum storm surge of record at Boston; extremely high onshore winds occurred during low astronomic tide and resulted in only a minor stillwater tidal flood level (7.6 feet NGVD).

Conversely, rather significant tidal flood levels can result from the coincidence of relatively high astronomic tides and relatively minor meteorological events. Astronomic high tide level in Boston alone can reach 7.4 feet NGVD (see table 7). With such a condition, a coincident storm surge of only 2 to 3 feet can produce major tidal flood levels. The 7 February 1978 storm tide at Boston reached 10.3 feet NGVD, the greatest of record, but was produced by a combination of astronomic tide of 6.9 feet NGVD and surge of 3.4 feet, the latter being of only moderate magnitude (see table 14 which shows that a surge of 3.4 feet is not extreme).

Windspeed observations recorded by the NWS at Boston's Logan Airport during the great blizzard of '78 are shown on figure 6. It shows gusts in excess of 55 knots (63 mph) for about four hours from the ENE. Average windspeeds were sustained above 43 knots (49 mph) for nearly four hours from the same direction.

d. Storm Tides. The total effect of astronomical tide combined with storm surge produced by wind, wave, and atmospheric pressure contributions is reflected in actual tide gage measurements. Since the astronomical tide is so variable at the study area, time of occurrence of the storm surge greatly affects the magnitude of the resulting tidal flood level. Obviously, a storm surge of 3 feet occurring at a low astronomical tide would not produce as high a water level as would be produced if it occurred at a higher tide. It is important to note that the storm surge itself varies with time, thus introducing another variable into the makeup of the total flood tide.

(1) Boston. The variation in observed tide and surge at Boston during the "Blizzard of '78" is shown in figure 7. It is interesting to note that the maximum surge (4.7 feet) occurred just before 10 p.m. on 6 February. However, the maximum observed tide occurred about 10:30 a.m. the following day when the surge had dropped by 1.3 feet. Had the maximum surge recorded during the storm occurred at 10:30 a.m. on 7 February, the observed tide would have been 11.6 feet NGVD, and would have resulted in even more catastrophic flooding at the project area. Annual maximum surge



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METROPOLITAN DISTRICT COMMISSION
ENVIRONMENTAL IMPACT REPORT
FOR THE
REVERE BEACH DEVELOPMENT PROJECT

TIDE LEVEL AND WINDSPEED VS. TIME FOR THE STORM OF FEBRUARY 6 AND 7, 1978.

Camp, Dresser & McKee, Inc.

Alan M. Voorhees & Assoc., Inc. Bolt Beranek & Newman, Inc.

FIGURE 6

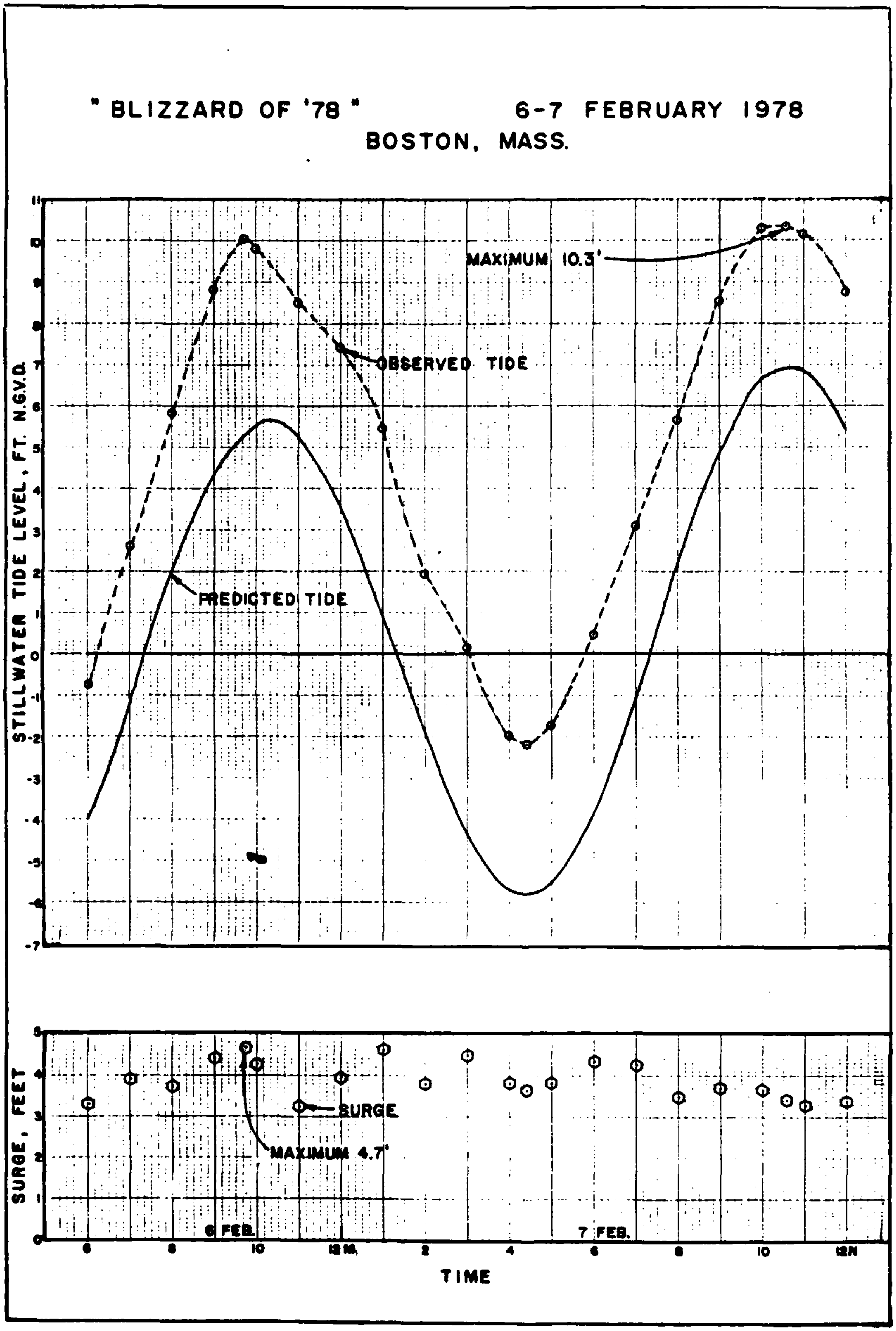

" BLIZZARD OF '78 "
6-7 FEBRUARY 1978
BOSTON, MASS.
STILLWATER TIDE LEVEL, FT. N.G.V.D.
MAXIMUM 10.3'
OBSERVED TIDE
PREDICTED TIDE
SURGE, FEET
SURGE
MAXIMUM 4.7'
6 FEB.
7 FEB.
TIME

FIGURE 7

values of greater than or equal to 3.0 feet measured at the Boston, Massachusetts, National Ocean Survey (NOS) tide gage are shown in table 14. The average annual maximum storm surge at Boston is 3 feet. This table shows the importance of coincident astronomic tide in producing significant tidal flooding (see the discussion in section 6c, which deals with the wind observations recorded during these events).

The NOS has systematically recorded tide heights at Boston, Massachusetts since 1922. The record prior to that time was developed utilizing staff gage measurements and historical accounts. Maximum observed stillwater tide heights (measurements taken in protected areas in which waves are dampened out) recorded through 1987 are shown in table 15. Also shown are the tide heights with an adjustment applied to account for the effect of rising sea level (see section 14). The greatest observed stillwater tide level recorded occurred during the "Great Blizzard of '78." No hurricanes or tropical storms are known to have produced extreme tide heights at Boston, thus indicating that historically the principal threat of flooding in the study area is due to storms of the extratropical variety.

(2) Study Area. Tidal observations throughout the study area made by NED and the USGS for the 2 January 1987 storm are compared to similar data from Boston in table 16. It can be seen that in general, for significant storm tides, water levels are less impacted by restrictions (i.e., bridge openings, I-95 embankment, etc.) than they are at normal spring tides. This can be seen by comparing the water levels of Fox Hill drawbridge and estimated water levels for Town Line Brook which show almost no difference. In table 10, the corresponding normal spring tide differences were approximately 0.5 foot lower.

TABLE 16

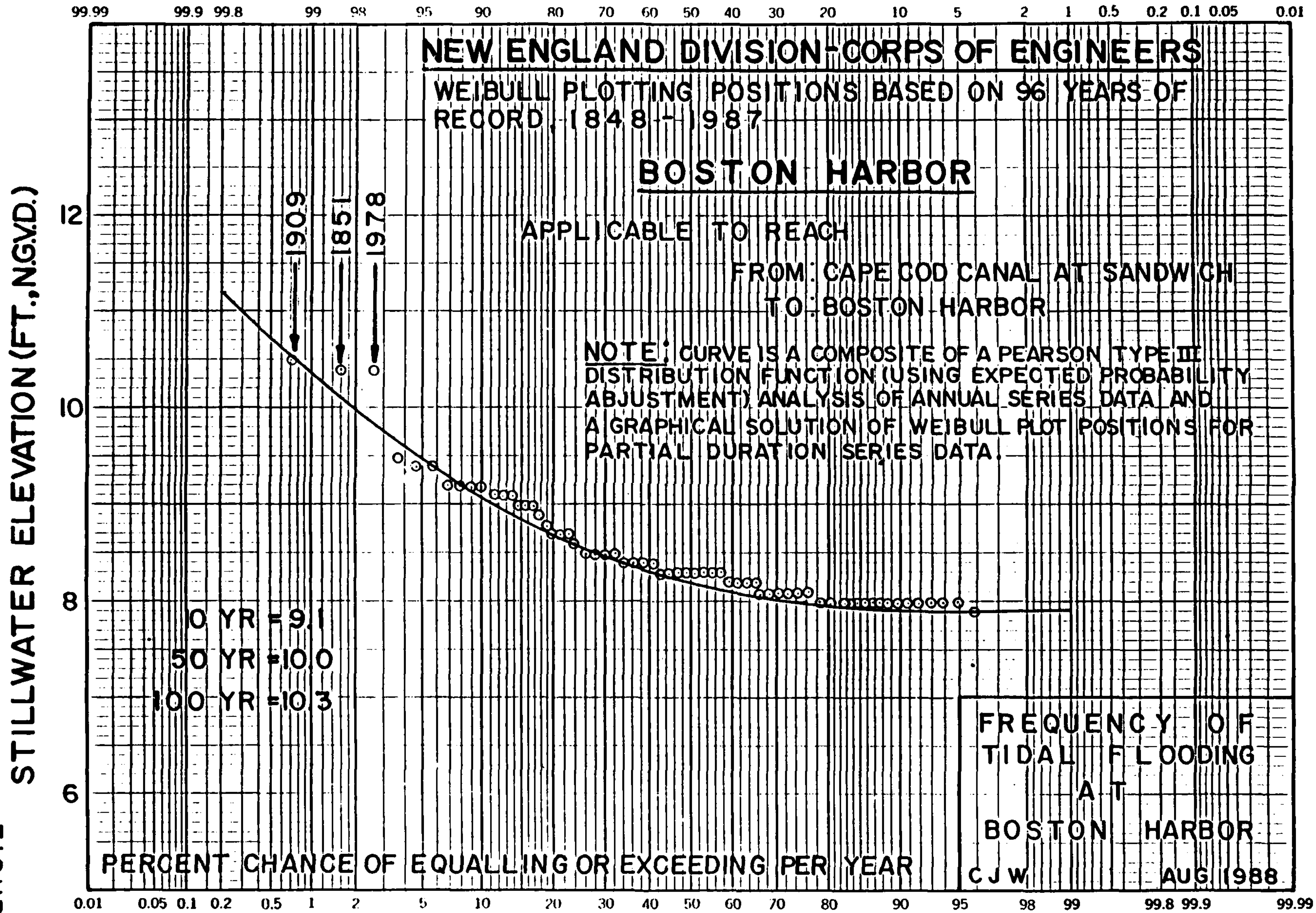
TIDAL OBSERVATIONS
2 JANUARY 1987 STORM EVENT

Location	Elevation (ft, NGVD)	Timing (hrs)	Source of Data
Boston	9.43	12.2	National Ocean Survey Gage
Saugus River at Fox Hill Drawbridge	9.57	13.0	USGS Gage
Pines River at Town Line Brook Tide Gate	9.5 (est)	--	High Water-Mark
Pines River Marsh at East Saugus	8.5 (est)	--	High Water-Mark

e. Tide-Stage Frequency

(1) Boston. A tide stage-frequency relationship for Boston was previously developed in 1979 utilizing a composite of (a) a Pearson type III distribution function, with expected probability adjustment, for analysis of historic and systematically observed annual maximum stillwater tide levels and (b) a graphical solution utilizing Weibull plotting positions for partial duration series data (reference: EM 1110-2-1412, 15 April 1986). A similar analysis of maximum stillwater tide levels was completed using updated tidal data obtained from the NOS through the year 1987. Comparison of the results, which take into account NOS' latest estimates of historic sea level rise, shows imperceptible differences from the 1979 curve and therefore a change is not considered warranted for this study. The resulting tide stage-frequency curve is shown on figure 8.

(2) Study Area. NOS tide gage records and high watermark data gathered after major storms have been utilized in the development of profiles of tidal floods along the New England coast. Additionally, profiles of storm tides for selected recurrence intervals have been developed utilizing tide stage-frequency curves and high watermark information. A location map and profile for the reach of New England coast bounding the project are shown in figures 9 and 10, respectively.



WITH 1 OUTLIER

FIGURE 8

FIGURE 9
BASE MAP FOR TIDAL FLOOD PROFILE

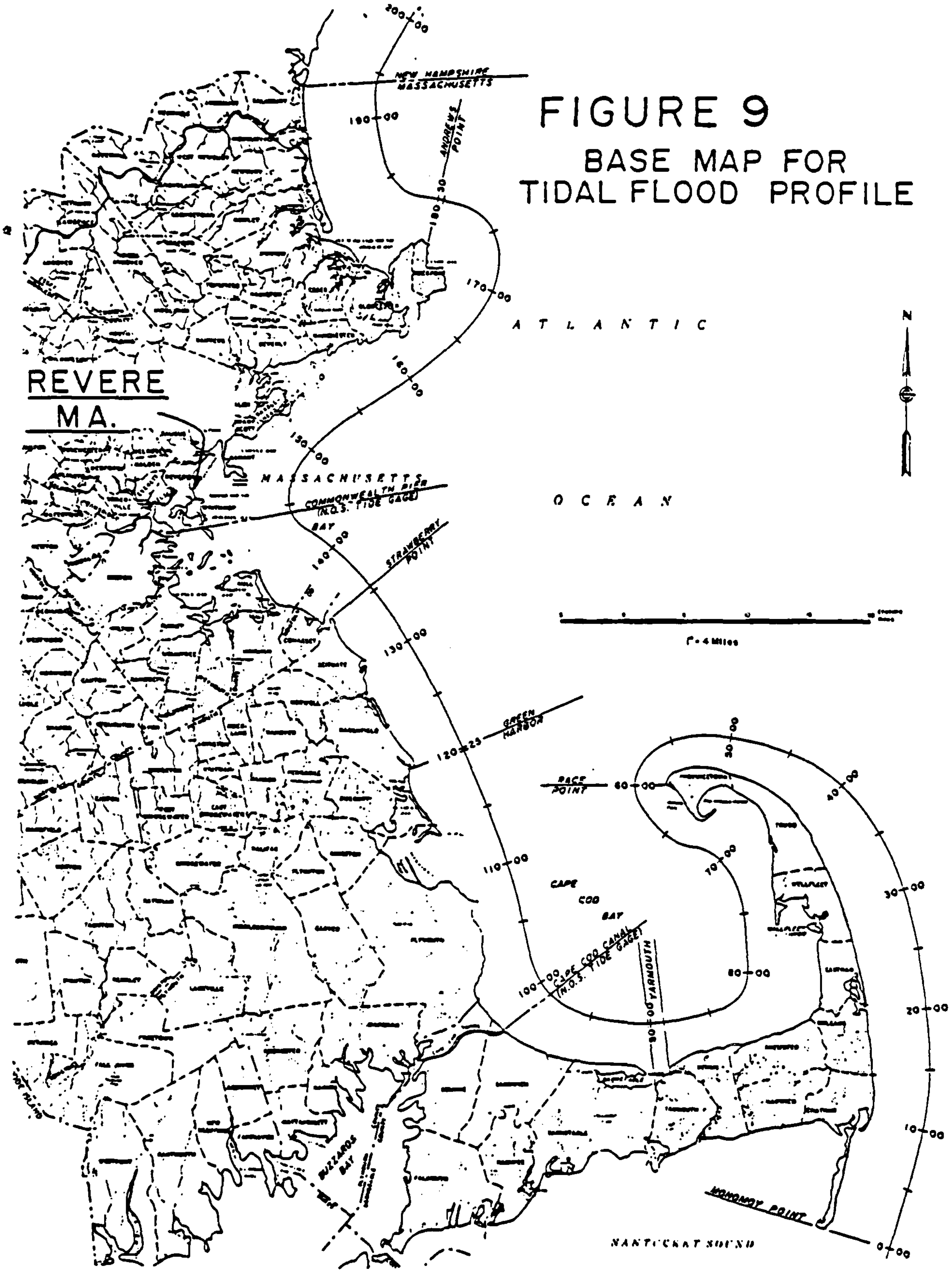
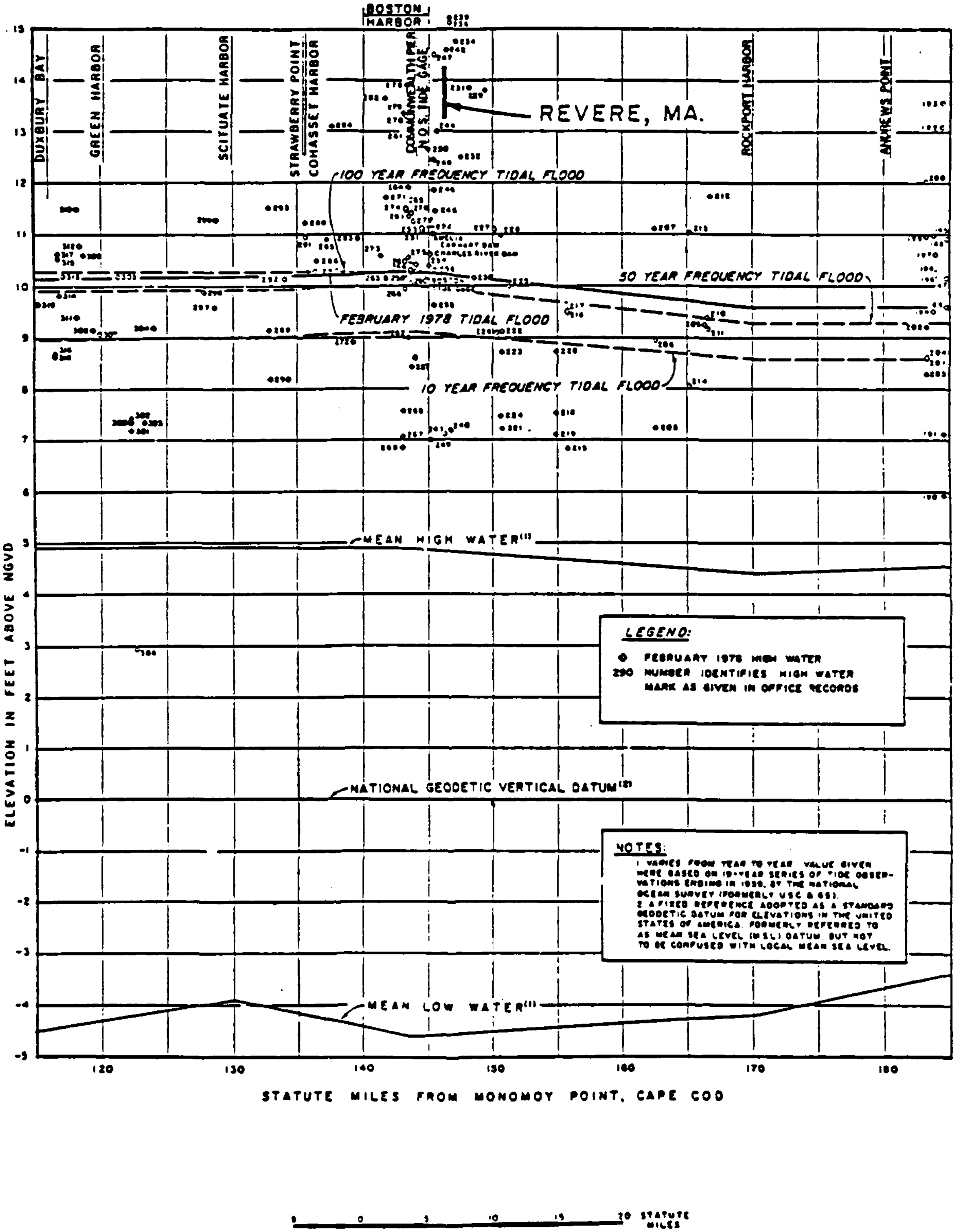


FIGURE 10

TIDAL FLOOD PROFILE



Additionally, studies by the Coastal Engineering Research Center (CERC) for the adjacent Roughans Point Project have indicated that storm tide frequency in the Saugus and Pines River system is nearly identical to that at the Boston NOS gage. CERC simulated surges for 50 randomly selected coastal storms (maximum water levels ranged from 7.9 to 11.2 feet, NGVD) using the WIFM model and determined their frequency. Results were presented in Technical Report CERC-86-8. During more frequent storm tide events, most built-up areas are protected by localized stretches of high ground, tide gates, etc. Therefore, information from the CERC report supplemented by knowledge of local drainage patterns and past flood high watermarks was used to develop flood stage frequency relationships for many separate damage zones. These curves and related explanation are contained in the following section.

7. TIDAL FLOOD PLAIN ZONES

a. General. Tidal flood plain zones susceptible to flood damages in the lower Saugus River Basin are generally those coastal reaches in Revere and Lynn which have direct ocean (Broad Sound) exposure and developed areas in Lynn, Revere, and Saugus, located on the periphery of the interior tidal estuaries. Existing condition flood elevation frequency estimates were made, for use by others, in determining flood damage estimates. Frequency relations were developed for 12 different hydrologic/hydraulic zones including some subareas within zones. Location of the 12 zones, six in Revere, three in Saugus and three in Lynn, and respective subareas are shown on plate 3. Development of the elevation frequency relationships for each zone and subarea was not a precise analytical process but involved consideration of the topographic and hydraulic features of each site, available information on historic flood levels, and strong reliance on the developed ocean stillwater elevation frequency relation discussed in the section 6e - "Tide Stage Frequency." In practically all cases the experienced February 1978 record storm tide level was assigned a one percent annual chance of occurrence based on the statistical analysis of long term storm tide records for Boston Harbor. The Boston record included adjustment of historical data for the gradual long term rise in ocean level.

b. Recent Tidal Flood History. Ocean storm accounts for the New England coast extend over a 300-year historic period and continuous records of ocean tides at Boston have been maintained since 1922. In that historic period, the greatest known tide level at Boston was 10.3 feet NGVD occurring in February 1978. Other historic events approaching that level, or exceeding after adjustment for rise in sea level, were 16 April 1851 and 26 December 1909, at elevation 10.1 and 9.9 feet NGVD and 10.4 and 10.5 feet NGVD after adjustment, respectively. No reliable systematic high water data is available in the lower Saugus River basin for the historic storm events. Resident recollection is generally limited to the previous 10- to 15-year period. Recent notable storm tides at Boston, in addition to February 1978, were:
2 January 1987 with a stillwater elevation of 9.4 feet NGVD,
25 January 1979 with a stillwater elevation of 9.3 feet NGVD,
and 19 February 1972 with an elevation of 9.1 feet NGVD.
Rainfall accompanying these recent highest tidal events is listed in table 3. Experienced high water elevations and developed flood stage frequency relations for selected flood plain zones in Revere, Lynn and Saugus, MA are listed in table 17.

TABLE 17

REVERE, MASSACHUSETTS
NATURAL AND MODIFIED
FLOOD STAGE FREQUENCIES
(Feet NGVD)

ANNUAL FREQUENCIES (%)

LOCATION - CONDITION		0.2	1.0	10	50	90	EXPERIENCED 1978	1979	1987	Others
BOSTON STILLWATER		11.2	10.3	9.1	8.2	7.9	10.3	9.3	9.4	9.1 (72) 8.9 (67)
ZONE 1 -	Natural	8.6	7.1	5.0	3.8	3.5	7.0	5.6	-	4.5 (72)
	Modified By Tidal Protec.	8.6	4.7	4.4	3.8	3.5				
ZONE 2A -	Natural	13.0	11.3	7.6	6.5	6.4	11.3	8.5	8.6	7.3 (67)
	Modified By Tidal Protec.	6.8	6.7	6.6	6.5	6.4				
ZONE 2B -	Natural	10.9	9.3	5.8	3.8	3.4	9.3	-	4.7	
	Modified By Tidal Protec.	4.8	4.6	4.4	3.8	3.4				
ZONE 3A -	Natural	8.0	6.1	5.0	4.4	4.1	6.4	-	-	
	Modified By Tidal Protec.	6.8	6.0	5.0	4.4	4.1				
ZONE 4A -	Natural	9.8	8.3	5.5	3.8	3.4	8.3	5.5	5.0 ±	
	Modified By Tidal Protec.	4.8	4.6	4.2	3.8	3.4				
ZONE 4B -	Natural	8.4	6.5	3.8	3.2	3.0	6.5	-	6.5 ±	
	Modified By Tidal Protec.	3.7	3.5	3.3	3.2	3.0				
ZONE 4C -	Natural	9.5	7.7	5.8	5.2	5.0	7.7	-	7.0 ±	
	Modified By Tidal Protec.	5.6	5.4	5.3	5.2	5.0				
ZONE 5A -	Natural	11.2	10.3	9.1	8.2	7.9	10.2	-	9.0	
	Modified By Tidal Protec.	7.6	7.4	7.2	7.1	7.0				
ZONE 5B -	Natural	11.2	10.3	9.1	8.2	7.9	11.0			
	Modified By Tidal Protec.	9.6	9.4	7.2	7.1	7.0				
ZONE 5C -	Natural	11.2	10.3	9.1	8.2	7.9	8.3	-	9.4	
AND 5D	Modified By Tidal Protec.	7.6	7.4	7.2	7.1	7.0				
ZONE 6 -	Natural	11.8	10.7	9.0	7.9	7.4	10.7		9.5	
	Modified By Tidal Protec.	7.6	7.4	7.2	7.1	7.0				

TABLE 17 (cont.)

LYNN, MASSACHUSETTS
NATURAL AND MODIFIED
FLOOD STAGE FREQUENCIES
(Feet NGVD)

ANNUAL FREQUENCIES (%)

LOCATION - CONDITION	0.2	1.0	10	50	90	EXPERIENCED 1978	1979	1987	Others
BOSTON STILLWATER	11.2	10.3	9.1	8.2	7.9	10.3	9.3	9.4	9.1 (72) 8.9 (67)
ZONE 1 - Natural	13.6	12.4	10.1	9.0	8.5	12.4	10.1	10.0 ±	
Modified By Tidal Protection	8.5	8.4	8.4	8.4	8.3				
ZONE 2 - Natural	12.0	11.2	9.9	9.1	8.8	11.2	-	10.0	
Modified By Tidal Protection	8.6	8.4	8.4	8.4	8.3				
ZONE 3 - Natural	11.2	10.3	9.1	8.2	7.9	10.3	9.3	9.6	
Modified By Tidal Protection	7.6	7.4	7.1	7.0	7.0				

SAUGUS, MASSACHUSETTS
NATURAL AND MODIFIED
FLOOD STAGE FREQUENCIES
(Feet NGVD)

ANNUAL FREQUENCIES (%)

LOCATION - CONDITION	0.2	1.0	10	50	90	EXPERIENCED 1978	1979	1987	Others
BOSTON STILLWATER	11.2	10.3	9.1	8.2	7.9	10.3	9.3	9.4	9.1 (72) 8.9 (67)
EAST SAUGUS									
ZONE 1 - Natural	12.1	11.0	9.2	8.2	7.9	11.0	9.2	9.6	-
Modified By Tidal Protection	7.6	7.4	7.1	7.0	7.0				
ZONE 2 - Natural	11.9	10.7	8.3	6.0 (21%)	-	10.7	-	7.2	-
Modified By Tidal Protection	6+	6-	-	-	-				
ZONE 3 - Natural (Upper Limit)	11.7	10.5	8.6	7.0	6.4	10.5	-	8.5	-
Modified By Tidal Protection	7.6	7.3	7.1	6.6	(6-)				

c. Flood Plain Zones - Revere

(1) Zone 1 - Crescent Beach Area, Revere: This zone is that coastal interior area of Revere extending north from Roughans Point (Eliot Circle) about 2,600 feet to Shirley Avenue. The area is triangular in shape generally bounded on the south by Sales Creek, on the west by North Shore Road, on the north by Shirley Avenue, and on the east by Crescent Beach (Revere Beach Boulevard). Historically, flooding in the zone has generally been limited to the interior low area generally west of Ocean Avenue and the MBTA tracks in the Garfield School Region and extending south to the Sales Creek. Minimum ground elevations in this interior area are generally around +4.0 feet NGVD. Drainage of the zone is generally to the south in a channel along the MBTA Railroad outletting to Sales Creek, which is a tidal estuary that originally drained west to the Chelsea River. However, the creek now drains west about 2,000 feet and then reverses direction draining southwest a distance of about 3,000 feet, through Suffolk Downs racetrack, discharging to the Belle Isle inlet of Boston Harbor at Bennington Street. There is a tide gate on the stream at Bennington Street. The capacity of the entire drainage system is a function of ocean tide. A plan of improvement for Sales Creek was developed by Andrew Christo Engineers for the Commonwealth of Massachusetts, Division of Waterways, in 1974. The plan includes channel and conduit improvements plus the construction of a pumping station and new tide gate at Bennington Street. The pumping station has been completed but little of the channel-conduit improvements have been accomplished. The completion of this plan and continued operation and maintenance of its extensive channel-conduit-station system, will result in an improved drainage outlet for the Crescent Beach area, provided improvements are made in its own local storm drainage system. Flooding in zone 1 is due primarily to limited storm drainage facilities, aggravated by high stages in the receiving Sales Creek. Under extreme tidal storms, tidal overtopping waters from zone 2 can enter zone 1 by flowing south from zone 2 along the MBTA Tracks. Based on field reconnaissance, it was concluded that tidal overtopping at Crescent Beach, opposite zone 1, in 1978 was quite minimal. An existing condition stage-frequency relationship for zone 1 in Revere was estimated based on reported average flood levels for the events of February 1978 - 7.0 feet NGVD, January 1979 - 5.6 feet NGVD, and February 1972 - 4.5 feet NGVD, and their associated storm tide frequencies at Boston. It was further concluded that for rare events, with tides appreciably in excess of 10 feet NGVD, there would be excessive overtopping of Bennington Street and resulting stages in Sales Creek and zone 1 would approach the Boston stillwater level.

(2) Zone 2: Wonderland, Revere: Zone 2 is the Revere Beach Wonderland Park area of Revere, generally bordered on the south by Shirley Avenue (zone 1), the B&M Railroad to the west, Revere Street on the north, and the Revere Beach seawall on the east. Subarea 2A (Police Station area) is the higher part of the zone generally east of North Shore Road and subarea 2B (Dog Track area) is the lowest portion located generally west of North Shore Road. Minimum ground elevations in subarea 2A are around +6.0 feet NGVD and in subarea 2B, as low as +3.0 feet NGVD. Drainage from 2A is to the west towards 2B where it combines with 2B drainage. This combined drainage flows first to the north about 2,400 feet, via "county ditch," crossing beneath Revere Street and then westerly about 600 feet, crossing beneath North Shore Road, then the B&M railroad tracks, and outletting to Diamond Creek just west of North Shore Road. Diamond Creek, in turn, is a tributary to the Pines River. There are tide gates on "county ditch" at North Shore Road and at the B&M embankment. There is a period of time during each 12-hour tide cycle when tide levels in the receiving Pines River prevent any gravity drainage from zone 2. During these periods any zone 2 storm drainage must be temporarily stored either by infiltration or surface pondage in the drainage system, adjacent low lying wetlands or parking lots.

It is noted that a tidal floodgate project at the mouth of the Saugus River (see paragraph 8b) could not be operated to maintain the Pines River low enough to provide continuous gravity drainage from low areas such as zone 2 and some residual local drainage problems, in the absence of tidal flooding, may persist. It would need to be stipulated, as a part of local assurances, that all existing flap gate structures outletting to the Saugus-Pines River estuary would be maintained in good operating condition, and any and all proposed new developments in these low areas would be thoroughly reviewed with regards to their potential impact on both drainage problems and/or needs.

Flooding can occur in subarea 2B as a result of either intense interior rainfall-runoff during periods of high tide or, more significantly, by waves overtopping the Revere seawall and flowing through subarea 2A into subarea 2B. In contrast, subarea 2A has reasonably good normal rainfall runoff drainage relief and serious flooding is primarily due to tidal overtopping of the seawall. A tidal overtopping flood stage frequency relationship was estimated for subarea 2A based on reported experienced 1978, 1979 and 1987 high water levels at the Police Station of 11.3, 8.5 and 8.6 feet NGVD, respectively. Start of significant overtopping was estimated to have about a 20 percent (5-year) annual chance of recurrence. The stage-frequency relation for subarea 2B was patterned after that for 2A except it was about 2 feet lower, considering the reported 1978 flood level (one percent chance) was about 9.3 feet NGVD, start of overtopping at about 20 percent (5-year), and allowing for some ponding from interior runoff in the more frequent range.

(3) Zone 3: Towle Area, Revere. Zone 3 in Revere is the westerly extension of subarea 2B, but is hydraulically separated from subarea 2B by the Boston & Maine (B&M) Railroad. Drainage in this area is easterly towards the railroad and then north along the railroad, first by ditch and then conduit, discharging to Diamond Creek and thence the Pines River. The drain is equipped with a tide gate at its point of discharge to Diamond Creek. Minimum ground elevations in the zone are around +3.5 feet NGVD. Drainage from the area is preempted for a period of time during each 12-hour tide cycle and flooding is generally limited to interior rainfall-runoff. Assuming the tide gate functions as designed, the area is considered quite free of tidal flooding until tidal flood levels in subarea 2B exceed the level of the B&M Railroad, which is at 10 feet NGVD. This would not occur until around the 0.2 percent chance (500-year) event according to the developed frequency relation for subarea 2B. The stage frequency relation for zone 3 was developed allowing for ponding of interior runoff (reported ponding in 1978 one percent chance event was 6± feet NGVD) and assuming no tidal flooding until the railroad is overtopped at about +10 feet NGVD (0.2 percent chance).

(4) Zone 4: Meadowlands, Revere. Zone 4 is that portion of the Revere Beach peninsula extending approximately 4,000 feet north of Revere Street and zone 2. Subarea 4A is between Revere Street to the south and Oak Island Street to the north and is bounded on the west and east by North Shore Road and Revere Beach Boulevard, respectively. Subarea 4B lies west of 4A and is the area between North Shore Road and the B&M Railroad. Subarea 4C is the area north of 4A extending about 2,000 feet north of Oak Island Street to the old narrow gage railroad embankment, bounded on the east by Revere Beach Boulevard and on the west by North Shore Road. An existing seawall along Revere Beach prevents any significant tidal wave overtopping into zone 4 and any flooding is due mainly to interior drainage and "back door" tidal overflow from the Pines River and its tributary - Diamond Creek. Drainage from subarea 4A is to the west to county ditch, which originates in zone 2 and continues west beneath the North Shore Road to subarea 4B. Zone 4B also drains to the ditch, on its westerly course beneath the B&M Railroad to Diamond Creek. Minimum elevations in subareas 4A and 4B are only about +2.0 to +3.0 feet NGVD. There is a tide gate both at North Shore Road (4A outlet) and at the B&M Railroad (4B

outlet). As was the case in zone 2, drainage from zone 4 is preempted during the high tide period of each tide cycle and drainage is infiltrated or temporarily ponded. Drainage from subarea 4C is to the north where it seeps beneath the North Shore Road to the Pines River. Minimum elevations in subarea 4C are +4 to +5 feet NGVD. In 1978 tidal wave overtopping into zone 2 flowed into zone 4 via county ditch plus there was "backdoor" overtopping of North Shore Road from the Pines River. As a result, ponding up to 6 feet deep occurred in subareas 4A and 4B and remained for up to a week due to clogged or limited drainage facilities. Subarea 4C did not reportedly receive flows from zone 2 but experienced flooding by Pines River overtopping of North Shore Road, as also occurred in 1987. Minimum elevations on the B&M Railroad are about elevation 9.0 feet NGVD, where as, minimum elevations on North Shore Road are more nearly +7.0 to +8.0 feet NGVD. Reported flood elevations in subarea 4A, used for estimating a stage-frequency relation, were ±8.2, ±5.5 and ±5.0 feet NGVD in 1978, 1979 and 1987, respectively. Peak elevations in the receiving subarea 4B were believed about 1.0 to 1.5 feet lower than 4A. Reported 1978 flood levels in subarea 4C were about elevation 7.7 feet NGVD, representing a 4-foot depth of flooding.

It is noted, as it was for zone 2, that a tidal floodgate project at the mouth of the Saugus River would not be operated to provide continuous gravity drainage from low areas in zone 4 and some residual local drainage problems, in the absence of tidal overflows, may persist. It would need to be stipulated, as a part of local assurances, that all existing flap gate structures outletting to the Saugus River estuary be maintained in good operating condition, and any, and all, proposed new developments in these low areas be thoroughly reviewed with regards to their potential impact on both drainage problems and/or needs.

(5) <u>Zone 5: Riverside-Oak Island and Vicinity, Revere</u>. Zone 5 is the remaining portion of the Revere Beach Peninsula lying north of zone 4 and outer Oak Island and continuing north about 7,000 feet to the mouth of the Saugus River, excluding the Point of Pines project area. Subarea 5A is the predominantly residential Riverside area lying between the Pines River on the west and North Shore Road on the east and extending about 3,500 feet south of the mouth of the Pines River. Subarea 5B is the low area to the west of Revere Beach Boulevard and to the east of North Shore Road. Subarea 5C is an area south of Riverside, bounded on the east by North Shore Road and on the west by the B&M Railroad. Subarea 5D is the periphery of Oak Island west of the B&M Railroad.

All drainage from zone 5 is west to the Pines River. Drainage from subarea 5A is mostly west to the Pines River with some to the east to a draw flowing south along the east side of North Shore Road. Drainage from subarea 5B enters this draw as it continues south and then turns west across the North Shore Road where it outlets to the Pines River. In addition to interior drainage, ocean wave overtopping of the ocean seawall in reach 5B, as experienced in 1978, flows west into the referenced draw and ponding area along North Shore Road. Start of seawall wave overtopping in reach 5B has been estimated as having about a 5 percent annual (20-year) chance of occurrence. It is noted that the drainage divide between 5B and reach 4C is an old abandoned railroad embankment crossing the draw between North Shore Road and properties along the Boulevard.

Drainage from subareas 5C and 5D is directly to the receiving Pines River. Minimum ground elevations in presently developed areas are around +7.0 feet NGVD and minimum elevations in draws and ponding areas are generally +4.0 to +5.0 feet NGVD. Frequencies of flood stages, above +7.0 feet NGVD in zone 5 were considered the same as those of the Pines River, which in turn were the same as the developed ocean stillwater tide frequency at Boston.

(6) Zone 6: Northgate Area, Revere. Zone 6 is located at the southern end, and on the periphery, of the Pines River tidal marsh and is comprised of properties east and west of Brown Circle and the Salem Turnpike. Flood-prone properties east of the circle are mostly residential. The commercial Northgate Shopping Center, located west of the circle, is generally on higher ground and received minor flooding in 1978. Minimum ground elevations in the developed area are around +7.0 feet NGVD. Reported 1978 flood levels in the area, averaged about 10.7 feet NGVD, or about one-half foot higher than the Boston stillwater level. Whereas, tide level measurements taken as part of this study, indicate the lower more frequent tides are about one-half foot lower than the Boston stillwater. The adopted stage-frequency relation for zone 6 was patterned after the Boston curve with adjustment in the upper and lower ranges.

(7) Point of Pines. See paragraph 8c of this appendix and the OCE approved Detailed Project Report, Point of Pines, Revere, MA, October 1984.

d. Flood Plain Zones - Lynn

(1) General. The flood plain in Lynn, fronting on Lynn Harbor or the Saugus River estuary, were divided into three zones. Limits of the three zones are shown on

plate 3. Following are brief descriptions of the three individualized, though hydrologically related, flood plain zones in Lynn.

(2) Zone 1: Coastal Area, Lynn. This zone is that exposed area of Lynn fronting on the ocean (Lynn Harbor). The zone has about 9,000 feet of frontage extending from the General Edwards Bridge northward to Lynnway Circle. The most exposed frontage is in the first 5,000 feet north of the bridge. The zone 1 flood plain is generally bounded by Lynn Harbor on the south and east and extends inland, to the west, as much as 2,000 feet and beyond the Route 1A highway. Much of the area has served in the past as a drive-in theater and large landfill with many commercial businesses along the Lynnway (Route 1A). A large commercial development is presently planned and under design for the landfill area. Minimum ground elevations in the area are about +8.0 feet NGVD. Drainage is to the east and southeast to Lynn Harbor, except there is a trunkline storm drain along Lynnway (Route 1A) draining south and discharging to the Saugus River at the General Edwards Bridge. In addition, there is a main 75-inch trunkline drain along Market Street near the northern end of zone 1, which drains a small area of high ground around Lynn Common. Exact details of whether local drainage (the area directly adjacent to Lynn Harbor) connect to this line are unclear. Detailed investigation of this area would be undertaken during design to determine if modifications to existing drainage system would be required. There is an existing bulkhead along the waterfront for about three-quarters of the length of this zone, with top elevations varying from 10 to 11 feet NGVD. Shallow flooding in the zone can result from intense rainfall-runoff with drainage limited by storm drainage facilities and high tides in Lynn Harbor. However, widespread flooding is generally the result of abnormal storm tides and accompanying wave overtopping, such as was experienced during the blizzard of 1978 and the storms of January 1979 and January 1987. Reported high water levels in the zone averaged 12.4 and 10.1 feet NGVD for the 1978 and 1979 floods, respectively. These flood levels, and their associated Boston stillwater frequencies were used as guidance in developing the adopted stage-frequency curve for this zone.

(3) Zone 2: General Electric, Lynn. The General Electric area in Lynn fronts on the Saugus River, between properties bordering the Lynnway and Western Avenue (Route 107) bridge, and extends inland about one-half mile. The General Electric complex covers an area of about 210 acres, consisting mostly of industrial buildings and paved areas. The storm drainage systems discharge south to the Saugus

River. Minimum ground elevations in the area are generally between +8 and +9 feet NGVD. Flooding in the area can result from intense interior rainfall-runoff in excess of drain capacity particularly when drain capacity is hydraulically impeded or preempted by high tide levels in the receiving Saugus River. During abnormal storm tides the area can also experience flooding directly from Saugus River and Lynn Harbors (zone 1) overflow and storm drain backflow. Reported high water levels in the area in February 1978 averaged about 11.2 feet NGVD. The adopted stage-frequency relation for this zone was developed based on the reported 1978 levels in the area relative to the computed Boston stillwater tide frequency relation.

(4) Zone 3: Saugus River, Lynn. This area is located along the left bank (north side) of the Saugus River between zone 2 and the Lynn town line. Its frontage on the river extends a tortuous distance of about 6,000 feet. It is developed with single family dwellings and some commercial businesses. All storm drains in the area discharge south to the Saugus River. Tidal flooding is generally the result of storm drain backup, during storm tides, and riverbank overflow. Reported high water levels for the February 1978 and January 1979 events averaged 10.2 and 9.3 feet NGVD, respectively. These elevations approximate the experienced Boston stillwater tide level and the Boston stillwater stage-frequency relation was adopted for this zone.

e. Flood Plain Zones - Saugus

(1) General. The flood plain zones in Saugus are comprised of three areas adjacent to the tidal estuaries of the Saugus and Pines Rivers. The three identified zones, though adjacent to each other, are separated hydraulically by road embankments Following are brief descriptions of the three zones and their hydraulic characteristics.

(2) Zone 1: Saugus River, Saugus. Zone 1 in Saugus fronts on the right bank of the Saugus River and is directly across the Saugus River from zone 3 in Lynn. The area is mostly residential, single family dwellings, with some commercial businesses. Storm drainage from the area is generally north to the Saugus River and hydraulic capacity is highly dependent on tide levels in the Saugus River. Reported high water levels in February 1978 and January 1979 averaged 11.0 and 9.2 feet NGVD, respectively. These flood levels, relative to the computed Boston stillwater frequency relation, were used in estimating a flood stage-frequency relation for this zone.

(3) Zone 2: Ballard to Bristow Street, Saugus. Zone 2 in Saugus is that area located west of the Salem Turnpike, extending from Ballard Street south to Bristow Street. Storm drainage from the area flows first east and then north to, and beneath, Ballard Street, outletting to the Saugus River. The drain culvert beneath Ballard Street is equipped with a flap gate. The tide gate prevents tidal backflow into the area unless, and until, tide levels exceed the elevation of Ballard Street. The low point elevation of Ballard Street is about elevation +7.8 feet NGVD. During periods of high tide all interior drainage must be temporarily ponded in the low areas south of Ballard Street. Minimum ground elevations in the area are ± 4 feet NGVD. Tidal flooding in the area is generally not excessive for tide levels in the Saugus River less than +8.0 feet NGVD. In February 1978 when the Saugus River was about 10.3 feet NGVD, flood levels in the area were reportedly from elevation 10.3 to 11.0 feet NGVD. The modifying effects of the flap gate, during moderate storm tides, the controlling elevation of Ballard Street, the experienced 1978 level, and peak stage frequencies on the Saugus River were all considerations in developing the estimated stage frequency relation for zone 2 in Saugus.

(4) Zone 3: West of Bristow Street, Saugus. Zone 3 is the most southwesterly area of Saugus fronting on the Pines River tidal marsh. The reach extends from Bristow Street south and then westerly along the marsh for a distance of about 3,000 feet to nearly Lincoln Avenue. The average

elevation of the marsh is about 5 to 6 feet NGVD, with drainage ditches, at lower elevations, meandering throughout. Lowest elevations in zone 3 are generally between +6.0 and 7.0 feet NGVD. Zone 3 is a residential area of about 125 acres. Most of the storm drainage discharges north to the Saugus River with some discharging south to the ditches in the Pines River marsh. Drainage of zone 3 is impeded by high tides in the Saugus River as well as the tide level in the marsh. Temporary ponding occurs in low areas and streets when intense rainfall occurs at times of normal high tides. Major flooding is caused by storm tides such as experienced in the February 1978 blizzard, which produced high water elevations surveyed at 10.5 feet NGVD. Development of the stage-frequency curve for this area was more complex than the other two zones in east Saugus due to its dependence not only on height of ocean tide but also on duration of high tide and coincident wind direction and magnitude. Two curves, representing threshold limits, were developed based on experienced 1978 levels and field observations during normal high tides in the Saugus and Pines Rivers.

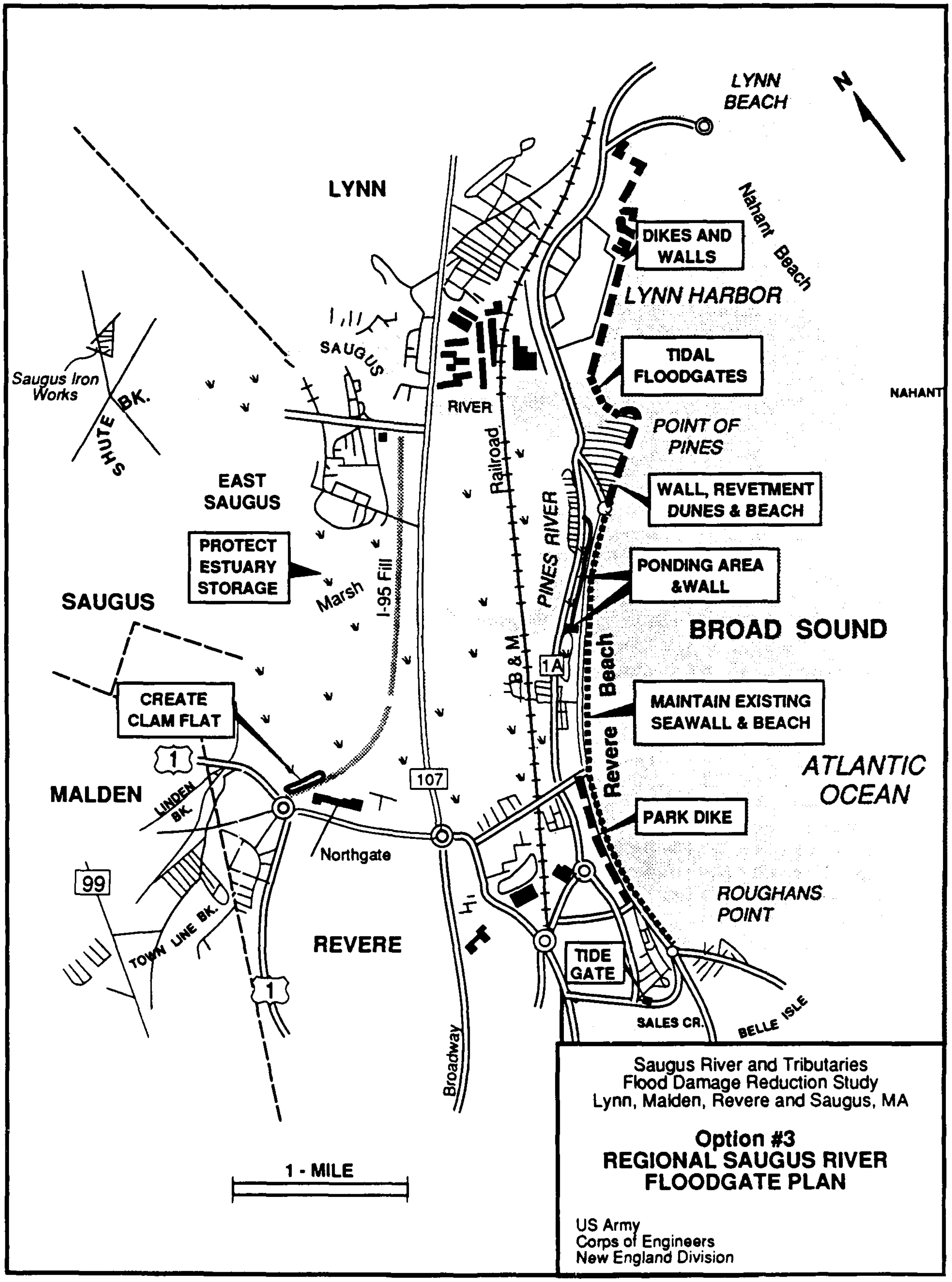
8. TIDAL FLOOD CONTROL

a. General. Two alternative structural plans considered for flood control and/or flood reduction in the study area were: (1) a system of individual local protection projects involving dikes and walls around individual localized damage zones, and (2) an integrated regional plan of protection, providing flood reductions at several damage zones, whose principal component would be a tidal gate structure at the mouth of the Saugus River. Both the local protection plans and integrated regional plan are described in the main report. Initial studies indicated that, of the two plans, the tidal regional floodgate plan had widespread public support and was economically and engineeringly feasible. Therefore, the following hydrologic discussion is directed principally at a regional plan of flood control. The Plan Formulation Appendix describes criteria and alternative alignments used to develop the local protection and regional plans.

b. Regional Saugus River Floodgate Plan. A Regional Saugus River Floodgate Plan (Figure 11) would include the following major Corps built components:

(1) A tidal floodgate across the mouth of the Saugus River with gated navigation and tidal flow passages.

(2) Connecting dikes and/or walls along Lynn Harbor in Lynn (fronting zone 1, Lynn).



Saugus River and Tributaries
Flood Damage Reduction Study
Lynn, Malden, Revere and Saugus, MA

Option #3
REGIONAL SAUGUS RIVER FLOODGATE PLAN

US Army
Corps of Engineers
New England Division

FIGURE 11

(3) An inland park dike in Revere (fronting zone 2A, Revere).

Additionally, local assurances would be necessary to preserve existing locally constructed protective works along Revere Beach, as well as preserving interior ponding areas and associated flap gates.

The regional plan would provide flood protection or reductions (see later discussions on project operation) in the three flood plain zones in Lynn, the three in Saugus, and in zones 2 through 6 in Revere. The Point of Pines area was also added late in the study. Tidal flooding in zone 1 in Lynn will be controlled by minimizing or eliminating tidal overtopping along the Lynn Harbor waterfront. Residual flooding would be limited to that produced by interior drainage. With the regional plan approximately 50 percent of this interior area would be permitted to drain behind the floodgate structure via existing storm drains. The remainder of the area, notably the northernmost section and areas directly adjacent to the line of protection, would continue to experience shallow temporary ponding in the event of high storm tide coincident with interior runoff. Tidal flooding in zones 2A, 2B, and 3A in Revere would be controlled by minimizing tidal overtopping at zone 2A and any residual flooding would be limited to that produced by interior drainage. Flood reductions in zone 1, Revere, attributable to the regional plan by eliminating inflows from zone 2B, would be quite limited. As part of plan formulation it was deemed infeasible to raise existing seawall protection along zone 5B, Revere. Therefore, existing tidal overtopping in this reach during severe storms will continue and allowance for such overtopping will be necessary in establishing interior storage capacity requirements in the estuary. Allowance will also be necessary for tidal overtopping at Point of Pines, with or without the separable Point of Pines Local Protection Project (see section 8c). Gate passages at a Saugus River floodgate would be designed and operated so as to have negligible effect on normal tidal flows and levels in the Saugus-Pines River estuary. Only during abnormal storm tides would the tidal floodgates be closed for regulating interior tidal levels (see discussion under project operations).

It is noted that the Regional Saugus River Floodgate Project could not be operated to provide continuous gravity drainage for those low areas, where interior drainage during normal high tides is presently dependent on localized ponding (zone 2 Saugus, zones 2B, 3A, 4A, and 4B Revere), and some residual local drainage problems, in the absence of tidal flooding, may persist.

It will be stipulated, as part of local assurances, that all existing flap gate structures discharging to the estuary be maintained in good operating condition and any, and all, proposed new developments in these low areas be thoroughly reviewed with regards to their potential impact on both drainage problems and/or needs. Project modified flood stage frequencies for the different zones and in the estuary depend on the magnitude of residual tidal overtopping and interior storm runoff during project gate closure. Computational procedures for establishing overtopping are reported under section 11e, "Wave Overtopping." Section 9 contains a discussion of interior runoff design considerations.

c. Point of Pines Local Protection Project. A separate Point of Pines Local Protection Project has been incrementally optimized by NED and was approved by OCE on 24 June 1985, the hydrologic and economic justification for which was presented in the Detailed Project Report, dated October 1984. The Division Engineer approved the inclusion of Point of Pines with the Regional Plan. Integration of the approved Point of Pines project with the regional floodgate plan (alignments 1 and 2 - see Plan Formulation Appendix) will affect economies to both projects. A section of the Lynn protection along the Saugus River as well as portions of the wall required along reaches F and G (near the Yacht Club along the Saugus River) for the Point of Pines Project can be eliminated. However, for the regional plans, a revetment or wall would be required in reaches E and F of Point of Pines, where failure of the sand dunes during rare events exceeding about a 1 percent chance could threaten the structural integrity of the regional floodgate plan.

As the decision was made to include the separable Point of Pines Local Protection Project into the Regional Plan, a review of the previously developed and approved stage frequency curves for Point of Pines (reference Detailed Project Report, Point of Pines, Revere, MA, October 1984 Appendix A) was considered necessary. From indepth coastal studies and modelling conducted by Waterways Experiment Station (reference Coastal Flooding, Roughans Point, Broad Sound, Lynn Harbor, the Saugus-Pines River System, T.R. CERC 86-8) stillwater tidal frequency elevations in the Saugus River have been refined and modified. This modification results in a change in previously developed elevation frequencies for the northern end of Point of Pines (zone 4 - see plate 5). Current estimates of applicable natural stillwater elevations in the Saugus River adjacent to the Point of Pines area together with previously developed elevation frequencies at zone 4 follow.

Percent Chance of Occurrence	Current Saugus River Stillwater Elevation (ft, NGVD)	Previously Developed Point of Pines DPR Zone 4 Elevation (ft, NGVD)
0.2	11.2	10.3
1.0	10.3	9.2
10	9.1	7.8
50	8.2	7.0

Because of the discrepancies in flood elevations, as can be seen above, and since zone 4 is affected directly by stillwater elevations in the Saugus River a review of this area was undertaken.

Available topographic mapping along Rice Avenue (adjacent to the Saugus River) shows that although Rice Avenue is low (6 to 7 feet NGVD), a concrete wall and high ground elevations lie between Rice Avenue and the river. Average elevations along the wall and high ground are approximately 8.5 to 9 feet NGVD. Therefore, with a Saugus River stillwater elevation of 8.2 feet NGVD (2-year frequency), it was determined that significant overflow into zone 4 would not occur and the previously developed elevation of 7.0 feet NGVD for zone 4 was adopted. With increasing storm intensity Saugus River overflow into zone 4 would begin to occur together with overtopping along reaches A to D. It is estimated that the one percent chance elevation in zone 4 would approach 10 feet NGVD or that of zone 3. Therefore, the existing condition elevations for the rarer events were modified slightly from the natural condition curves, reported in the Point of Pines DPR, for zone 4. Recommended revised existing condition elevation frequencies for zone 4 are shown below.

Percent Chance of Occurrence	Recommended Revised Zone 4 Elevation (ft, NGVD)
0.2	11.0
1.0	9.8
10	8.0
50	7.0

As requested by OCE, in their comment on the DPR, dated 2 April 1985, NED conducted a review of overtopping estimates for the design condition presented in the DPR based on more detailed tidal information and WES modelling studies. It was determined, based on updated overtopping volumes, that no significant change to the previously developed modified and natural stage frequency curves for zones 1 through 3 of Point of Pines would occur. However, by constructing a tidal floodgate at alignments 1 or 2 the receiving Saugus River elevation at the north end of Point of Pines would be reduced for storm tides. Therefore, a review of previously developed modified stage-frequency curves in Point of Pines was made to determine any effect of a lower Saugus River elevation. The optimized level of design at Point of Pines presented in the DPR's H & H Appendix called for upgrading or constructing protective works along reaches A, B, C, and D. This would significantly reduce overtopping at the 1 percent chance event; however, uncontrolled overtopping begins occurring with floods in excess of a 1 percent chance. The previously developed modified interior stage frequency curves for the Point of Pines zones are considered applicable up to about the 1 percent chance event. Up to this point the lowered receiving Saugus River elevation would have little impact on interior levels. With floods exceeding a 1 percent chance, when major overtopping begins, the lowered tailwater elevation would provide for improved drainage throughout the area. Overtopping waters would flow from south to north along the natural drainage course in Point of Pines outletting in the Saugus River. With the lowered Saugus River elevation and the wall and high ground along Rice Avenue at around 8.5 to 9.0 feet NGVD the estimated modified elevation in zones 3 and 4 for the 0.2 percent chance (500-year) event would be around 10 to 10.5 feet NGVD. With floods exceeding a 500-year event it is not expected that elevations would rise much over 11± feet NGVD.

In zones 1 and 2 with minimum Lynnway elevations of 11.5 to 13± feet NGVD it is not expected that there would be any indicated change from the modified curves presented in the DPR.

With the floodgate at alignments 1 or 2 and walls constructed or upgraded to 3 feet above design stillwater elevation from the floodgate structure along reaches E and F, overtopping along these areas would be prevented. Therefore, under this condition, hydraulics of the interior area would be modified and future condition zone limits would be changed. Modified zone limits for zones 2, 3, and 4 for the above described condition are shown plate 5. It is noted for economic purposes that Point of Pines zones 1 through 4 have been renamed 7A through 7D, respectively.

d. Town Line Brook - Malden. Town Line Brook (see plate 1) currently has flap gates on its outlet to the estuary beneath Route 1; therefore, tidal flooding is prevented, except during unusual storm events when some shorefront overtopping occurs. Flood problems are caused primarily by interior runoff which ponds in and along the brook during a normal high tide when the flap gates are closed. The Regional Floodgate Project cannot be operated to enhance drainage from this area during a normal high tide. Any improvement to the existing interior drainage problems attributable to the Regional Floodgate Plan would not be significant.

The principal benefit of the Regional Floodgate Plan would be the prevention of unusually high storm tides from overtopping the existing shorefront areas thereby decreasing flood levels within the interior of Town Line Brook.

e. Shute Brook - Saugus. The flood reduction to this area attributable to the Regional Floodgate Plan would be principally limited to preventing high storm tides in the lower (tidal) section of the brook below Central Street. Incidental reductions could occur further upstream, between Central and Vine Streets, during the infrequent occurrence of high watershed runoff coincident with a high storm tide. It was not attempted to quantify potential reductions for this hydrologic situation. Flood stages upstream along Shute Brook are considered a function of the hydraulic characteristics of the brook channel and any restrictions to flow such as undersized culverts or bridges.

9. INTERIOR RUNOFF DURING STORM TIDES

a. General. With a Regional Saugus River Tidal Floodgate Project, probable modified flood level frequencies in the estuary would be a function of probable volume of inflow during periods of gate closure for storm tides. Consideration was given to this runoff in estimating probable modified interior flood level frequencies. However, potential interior runoff during storm tides was also considered in assessing project design capability and project operating procedures.

b. "Probable Interior Runoff Coincident with Storm Tides." Methods for analyzing coincident interior runoff frequencies are presented in Corps of Engineers EC 1110-2-247 "Hydraulic Analysis of Interior Areas" dated 23 September 1983. If the occurrence of tidal and interior storm runoff events were completely independent, then an inflow duration curve (runoff rate versus percent of time) could be developed and this curve separated into a series of percent chance runoff events, i.e., ten different runoff rates for each 10 percent time duration. The 10 different runoff rates could then be used to compute ten different resulting interior storage levels and these levels could be averaged to arrive at the most probable average interior storage level. For further discussion of this approach reference is made to EC 1110-2-247.

The above method is most applicable to the analysis of probable coincidence of completely independent meteorological events. However, storm tides and rainfall-runoff are not entirely independent. Storm tides are generally associated with coastal storms and/or hurricanes with accompanying rainfall. Therefore, it would be expected that runoff-duration during periods of storm tides would be somewhat greater than that represented by the normal 100 percent of time runoff-duration. An adjusted storm tide runoff-duration curve was developed by determining recorded runoff rates at nearby gaged streams for a series of storm tide periods. It was found that runoff (streamflow) during storm tides, on average, was about 8 times greater than the all time average runoff (streamflow). Comparative flow durations for the gaged Old Swamp River in South Weymouth, Massachusetts (D.A. = 4.5 square miles) for 100 percent of time and for times of storm tides are shown on plate 4. The higher storm tide flow duration was used to estimate probable streamflow coincident with storm tides. The 10 percent of time storm tide flows were applied, by a drainage area ratio, with the 10 percent chance Boston tide level. Similarly the 1 percent of time storm tide flow was applied with the 1 percent chance storm tide in computing estimated most probable coincident interior runoff and resulting probable modified interior flood level frequencies.

c. Potential Runoff Coincident with Storm Tides. In addition to probable interior runoff, potential storm runoff coincident with storm tides was considered for establishing project design capability and project operating procedures. In assessing potential runoff, the runoff from the upper Saugus River Basin above tide water (25.7 square miles) was treated as one component and local runoff from the lower 21 square miles of local urban areas and water bodies was treated as a second component, with total inflow to the estuary being the sum of the two components plus any residual tidal overtopping. In selecting a potential interior runoff for project design and project operating procedure, the severity of the runoff was varied in proportion to the severity of project design tide. For a 0.2 percent chance (500 year) tide design a coincident 2 percent chance design peak interior runoff rate was assumed. Whereas, for a 1 percent chance (100-year) tide a 10 percent chance peak interior runoff was assumed, etc. Further discussions of the application of "potential" interior runoff in analyzing interior storage requirements and project operations is presented in paraparagraphs 12b - "Design Interior Runoff Criteria" and 13 - "Project Operation."

10. HYDROLOGIC ANALYSIS FOR PROJECT DESIGN OPTIMIZATION

Modified interior flood stage frequencies were developed for each flood plain damage zone, for use by planners in computing project benefits for different design levels of protection. Average annual project benefits were the difference in damages between preproject natural and postproject modified stage frequencies for different levels of design. The modified interior flood level was a function of interior runoff into storage (the higher the tide the longer the duration of interior runoff into storage), plus any tidal overtopping into the flood plain zone. Modified storage level frequencies were computed using "probable" interior runoff and start of storage was assumed at elevation 7.0 feet NGVD average in the estuary though gate closure level would vary depending on forecasted tide level, as discussed in paragraph 13. Starting storage in the tide gated areas was based on minimum ground elevations in the respective areas.

The overall optimum project design level was sought for all nonseparable components of the regional flood control plan. The separable Point of Pines Local Protection Project was incrementally optimized during past studies. Hydrologically it was recommended to project formulators that all benefits be taken at 100 percent up to the project level of design, and any benefits indicated for floods in excess of project design, be taken at 50 percent, generally in accordance with criteria for flood damage benefits in the freeboard range, presented in EP 1105-2-45, "Economic Considerations," dated January 1982. Modified flood stage frequency curves were developed for each flood plain zone for three alternative project levels of design: 1 percent (100-year), 0.2 percent (500-year), and standard project northeaster (see SPN discussion in paragraph 11a). A representative set of natural and modified flood stage-frequencies are shown on plate 6. Existing and modified (SPN Design Level) elevation frequency data for all zones are shown in table 17. It is noted that the SPN Design Level is the selected plan.

11. TIDAL HYDRAULICS

a. Standard Project Northeaster (SPN)-Tide Level. Previous analysis conducted during the feasibility investigation for the adjacent Roughans Point project resulted in an estimated ocean stillwater tide level of 13.0 feet NGVD for the SPN. OCE approved use of this estimated value pending formal development of the SPN tide level (reference: DAEN-CWE-H, 17 November 1980, 1st Ind, "Hydrologic Criteria - Revere, Massachusetts Coastal Flood Protection"). During a subsequent meeting between NED, WES, and OCE it was agreed that a less formal analysis of the SPN would be conducted by WES for use in the Revere area along with physical and mathematical modelling of wave overtopping at Roughans Point (reference: DAEN-CWH-Y, 5 March 1984, 1st Ind and NEDED-WQ, 11 April 1984, 2nd Ind, "Hydrologic Criteria - Roughans Point Coastal Flood Protection, Revere, Massachusetts"). The following presents a discussion of the WES evaluation of the Standard Project Northeaster Tide Level.

The Standard Project Northeaster (SPN) definition can be determined from the definition for the Standard Project Storm (EM 1110-2-1411) as the northeaster which results from the "most severe combinations of meteorologic and tidal conditions that are considered reasonably characteristic of the geographical region involved, excluding extremely rare combinations." For this report two processes are important in considering the specification of an SPN, stillwater level and wave overtopping. It is possible that a separate SPN would have to be defined for each process. The SPN which would produce the highest ocean stillwater level might not produce the highest waves in the study area and therefore, not the highest overtopping rates.

The SPN stillwater level was estimated to be 13.0 feet, NGVD in NED's feasibility studies by adding together the maximum surge recorded at Boston, about 5 feet, and the maximum predicted astronomic tide, 7.5 feet, NGVD, and then rounding up to the next foot of elevation. This resulted in a stillwater ocean elevation which was almost 3 feet higher than the maximum ever recorded at the Boston gage. Given the unlikely event that a tide with a maximum elevation near the maximum predicted astronomic tide were to occur sometime during the maximum surge-producing northeaster, the probability that the hour of maximum surge (using hour increments) would occur at the hour of maximum tide is only 1/24 (assuming a semidiurnal tide with unequal highs). Consequently, this combination might fall under the "excluding extremely rare" clause in the definition of the SPN. A better specification of the SPN stillwater level might be closer to 12.0 feet, NGVD. Therefore, for purpose of this report, a revised SPN stillwater tide level of 12 feet, NGVD has been adopted.

b. Policy on Design Level of Protection. The criteria used in establishing top elevation of coastal protection and determining degree of flood protection in the Saugus River and Tributaries Study is as follows:

(1) Protection Not Designed to Withstand Wave Overtopping (Sand Dunes, Earthen Dikes, Etc.)

(a) The height of protection and its integrity will be such that no wave overtopping will occur during the design flood. The top elevation of protection will be set one foot above the maximum runup (due to one percent chance wave, H_1) as measured from design stillwater level. At a minimum, the height of protection will be 3 feet above design stillwater level.

(b) In assessing project benefits, the protection will be considered fully effective up to the design flood and will be considered 50 percent effective for floods exceeding the design flood until any wave overtopping begins. When significantly overtopped the project is assumed ineffective.

(2) Protection Designed to Withstand Wave Overtopping (Concrete Walls, Fully Riprapped Dikes, Etc.)

(a) In the absence of interior drainage improvements (pumping, etc.), the height of protection will be set at the level of significant runup (due to the significant wave, H_s or H_{mo}) as measured from design stillwater level. A lower height of protection may be used if interior drainage improvements are included to provide full protection up to the design flood event. In no case shall the top elevation of protection be less than 2 feet above design stillwater level. Wave overtopping for events exceeding the design flood will be included in residual interior flood analysis.

(b) In assessing project benefits, the protection will be considered fully effective up to the design flood and, in the absence of detailed analysis or a physical model, will be considered 50 percent effective for floods exceeding the design flood until the structure is overtopped by the stillwater level, after which the project is assumed ineffective.

(3) Height Allowance for Uncertainty. It is normal engineering practice to include a minimum height allowance

above design stillwater level in the design of any flood protection project. This allowance is included to account for uncertainty in determining design conditions. The ocean is a fierce adversary with wind, surge, and wave conditions varying from point to point along the coast. There is an inherent uncertainty in determining the exact magnitudes of the design water level. Additionally, sea level has historically been rising at a rate of about 1-foot per century. Some experts have recently predicted an accelerated increase in sea level in the future. This introduces additional uncertainty in defining design conditions. It is felt that prudent engineering dictates a minimum allowance of 2 feet to account for uncertainty in the design of protection which can accommodate some wave overtopping. Where the protection is not designed for wave overtopping, this minimum allowance should be increased to 3 feet.

(4) Operational Considerations. In the case of the floodgate proposed for the mouth of the Saugus River, it is important to realize that the crest of the structure should be set high enough above design stillwater to assure trouble-free operation of gate hoisting mechanisms. For typical severe wave conditions at the structure (H_s=2.4 feet, T_s=2.9 sec), slightly over 3 feet of wave runup would be expected (see section 11d). Therefore, some degree of spray overtopping may occur but it should not be of great enough magnitude to adversely affect project operation.

(5) FEMA Criteria. Although not directly applicable to Corps of Engineers projects, it is useful to examine the criteria used by the Federal Emergency Management Agency (FEMA) in determining if a flood protection project is considered effective for flood insurance purposes. FEMA criteria (CFR, Vol 51, No. 60, 28 March 1986) states that: "for coastal levees, the freeboard must be established at one foot above the height of the one percent wave or the maximum wave runup (whichever is greater) associated with the 100-year stillwater surge elevation at the site." Their criteria goes on to say that occasionally an engineering analysis relating to uncertainty, wave attack and overtopping may justify an exception. However, "under no circumstances. . .will a freeboard of less than 2 feet above the 100-year stillwater elevation be accepted." It is felt that the 2-foot minimum freeboard criteria as proposed by FEMA is a reasonable absolute minimum in the coastal zone.

c. Wave Height. A steady-state, shallow water, directional-spectral wave model (ESCUBED) was previously used in studies for the adjacent Roughans Point project to simulate waves in the Broad Sound area (Technical Report CERC-86-8). For each of 50 randomly selected storm events (surge plus tide) with maximum water levels ranging from 7.9 to 11.2 feet, NGVD, the wave climate in a 25.9-square mile area of Broad Sound was simulated for each hour when the stillwater level was above 7.0 feet NGVD. Directional-spectral wave train characteristics from the Wave Information Study (WIS Report 9) were used along the eastern boundary to drive the model.

Resulting wave heights in the lee of Nahant Peninsula indicated that local wave generation in this area was inadequately simulated by ESCUBED. Hence, an additional analysis was required when winds were from the northeast. Shallow water wave growth equations were used to estimate locally generated wave heights and periods off the north seawall at Roughans Point as well as at Point of Pines and in Lynn Harbor. The total wave climate in these regions was assumed to be a combination of these locally generated waves and ESCUBED results.

It is important to note that no wave data collected from Broad Sound were available. Hence, it was not possible to calibrate ESCUBED or to verify its results. However, it is the best information available at the present time.

Table 18 is a summary of the wave heights, periods, and directions from the ESCUBED modeling for several locations from Roughans Point up along Revere Beach to Point of Pines. These locations are marked 1 through 9 in figure 12. Table 19 is a summary of the locally generated waves which were estimated using previously discussed Boston wind data. These areas are marked A through E in figure 13. These two tables are provided to demonstrate the range of wave parameters generated by the models. Waves were modeled only during periods of possible overtopping at Roughans Point (water levels above 7.0 feet NGVD) during northeaster conditions. The average values shown do not take into account the varying probabilities of the surge-tide-wave events.

TABLE 18

SUMMARY OF ESCUBED WAVE PARAMETER RESULTS

Location*	Height, Ft** Min.	Avg.	Max.	Period, Sec Min.	Avg.	Max.	Direction, Deg From North Min.	Avg.	Max.
1	0.5	5.9	9.6	1.9	9.3	14.3	30	92	97
2	0.2	1.9	3.4	1.7	7.9	14.3	**	**	**
3	0.4	5.4	9.5	1.9	8.9	14.3	31	91	103
4	0.2	3.9	8.7	1.7	9.1	14.3	34	109	111
5	0.2	5.1	9.6	2.0	9.1	14.3	100	114	149
6	0.2	4.5	9.1	2.0	9.1	14.3	100	122	149
7	0.2	3.4	8.0	2.0	9.1	14.3	100	130	149
8	0.2	1.8	4.3	2.0	9.1	14.3	100	145	149
9	0.2	0.2	0.3	1.9	3.8	4.3	100	141	150

Notes: *Refer to figure 13 for locations.
**Wave height for the north wall at Roughans Point was calculated from the two direction bands (70 and 50 deg) which were the closest to perpendicular to the north wall. No direction was calculated for these waves.

TABLE 19

SUMMARY OF LOCALLY GENERATED WAVE RESULTS

Location*	Height, Ft*** Min.	Avg.	Max.	Period, Sec Min.	Avg.	Max.	Direction, Deg From North Min.	Avg.	Max.
A	0.3	1.8	3.7	1.2	2.3	3.3	0.0	31.4	67.5
B	0.1	1.1	2.4	0.6	1.6	2.9	0.0	41.0	202.5
C	0.4	1.3	2.4	1.2	1.9	3.0	67.5	84.3	225.0
D	0.3	1.2	2.4	1.1	1.8	3.0	67.5	84.6	225.0
E	0.3	1.0	2.4	1.0	1.6	3.0	67.5	84.3	225.0

Notes: *Refer to figure 12 for location.
**H_{mo}
***H_s

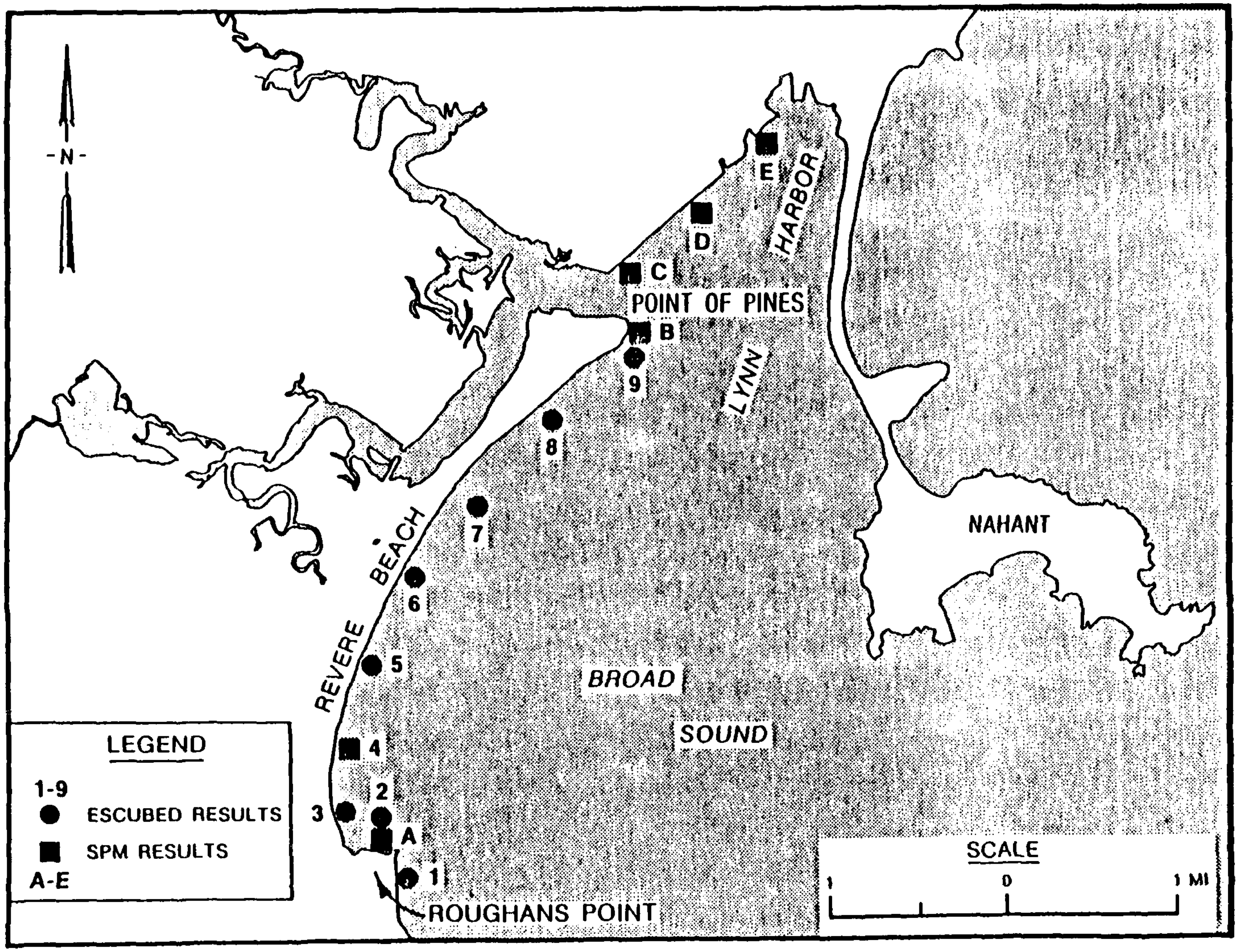


Figure 12 Locations for which locally generated waves were calculated

d. Wave Runup. Wave runup estimates were developed using the Corps Shore Protection Manual (1984) for the proposed structures to be constructed as part of the Corps project. In addition, runup values were determined for the existing coastal flood protection structures that will not be modified as part of this project along Revere Beach (these structures are described in the Main Report). In general, any runup height greatly exceeding top of the structure constitutes a significant overtopping condition, which could eventually flow into the backshore areas and into the tidal estuary. Estimates of wave heights described in the previous section have been used or modified slightly based on local conditions to determine wave runup; results for three different design conditions (100-year, 500-year and SPN) are presented in tables 20 through 22. Locations of the areas evaluated are shown in plate 10. In general, proposed structure configuration and the associated runup values remain the same for both the LPP option and regional plan option, with the exceptions being reaches C5 and D of the Revere Beach area where height of the wall had to be raised to further reduce overtopping for the LPP option.

It should be noted that wave runup values had previously been developed in the Point of Pines area as part of the OCE approved Point of Pines Flood Protection DPR, dated October 1984. However, as a result of the more comprehensive study of waves and overtopping phenomena associated with seawalls and revetments undertaken as part of the WES reports on Coastal Flooding at Roughans Point in September 1986, a reevaluation of runup and overtopping quantities using revised information was recommended. Runup values for this area are shown for the modified conditions as described in the DPR. For the Revere Beach area, it was assumed that the beach configuration as it exists now would be the future condition, even though the proposed Revere restoration project would significantly reduce the height of runup during frequent storms if the beach is properly maintained. Proposed design model studies of the beach are discussed in the Design and Cost Appendix. For the Revere Beach area, local assurances should be obtained to maintain, as a minimum, conditions as they exist along the non-Corps constructed areas (this is further discussed in the Plan Formulation Appendix).

e. Wave Overtopping. Estimates of wave overtopping were computed for the coastal project area from the south end of Revere Beach (reach A) to the north end of the Lynn bulkhead and the results for three different design conditions (100-year, 500-year and SPN with associated maximum storm tide levels of 10.3, 11.2 and 12.0 feet, NGVD) for the regional plan option are shown in tables 20 to 22. In general,

TABLE 20

WAVE RUNUP AND OVERTOPPING - 100-YEAR DESIGN - REGIONAL PLAN

Structure	Top Elevation** (ft. NGVD)	Ground At Toe (ft. NGVD)	Structure Slope V:H	Length Of Structure (ft)	Maximum Significant Wave (ft)	Maximum Significant Runup: Maximum Height (ft)	Maximum Significant Runup: Elevation (ft NGVD) 100-Year Event	Maximum Significant Runup: Elevation (ft NGVD) 500-Year Event	Maximum Significant Runup: Elevation (ft NGVD) SPN Event	Overtopping*** (ac-ft) 100-Year Event	Overtopping*** (ac-ft) 500-Year Event	Overtopping*** (ac-ft) SPN Event
Floodgate Structure												
Concrete	13.3	-14 to -18	Vertical	600	2.4	3.3	13.6	14.5	15.3	0	10	110
Embankment	13.3	-14 to 9	1:2	700	2.4	3.1	13.4	14.3	15.1	0	10	110
Lynn Bulkhead												
Reach A	13.0	-3	1:2	700	2.4	3.3	13.6	14.5	15.3	1	8	85
Reach B	15.0	-3	1:2	1800	3.4	4.3	14.6	15.5	16.3	0	0	5
Reach C	13.3	-3	1:2	2000	2.4	3.3	13.6	14.5	15.3	1	22	290
Reach D	13.3	-3	1:2	2550	2.4	3.3	13.6	14.5	15.3	1	5	140
Reach E	12.3	-3	1:2	1300	2.1	2.8	13.1	14.0	14.8	1	55	3995
Reach F	13.3	-3	1:2	1200	2.1	2.8	13.1	14.0	14.8	0	1	40
Revere Beach												
Reach A1	16.2	14.5	Vertical	1430	10.5	2.0	12.3	13.2	14.0	0	0	0
Reach A2	19.3	12.5	Vertical	525	9.5	2.7	13.0	13.9	14.7		0	0
Reach B1	18.4	6	Vertical	1475	7.0	13.8	18.7	21.6	26.8	[illegible]*	120*	300*
Reach B2	20.4	1	Vertical	570	9.5	23.1	28.4	30.9	33.0	100*	205*	340*
Reach B3	16.8	3	Vertical	1515	9.0	7.5	17.0	18.5	19.5	460*	825*	1250*
Reach B4	18.0	8.5	Vertical	545	7.3	19.2	23.7	26.5	30.0	105*	240*	500*
Reach C1	16.9	13	Vertical	1355	8.6	3.9	13.7	14.7	15.9	0	0	0
Reach C2	20.4	12	Vertical	565	8.6	3.9	13.7	14.7	15.9	0	0	0
Reach C3	16.1	12	Vertical	1360	8.6	2.4	12.5	13.5	14.4	0	0	0
Reach C4	15.9	12	Vertical	1300	10.4	2.8	12.9	13.9	14.8	0	0	0
Reach C5	16.0	3	Concrete Steps	660	5.8	12.8	21.4	22.3	24.8	50	105	230
Reach D1	16.0	3	Concrete Steps	1480	5.8	12.8	21.4	22.3	24.8	200	395	875
Reach D2	16.0	8	Vertical	900	2.6	8.1	16.4	17.9	20.1	5	20	60
Point of Pines												
Reach A	16.5	5	1:3	230	4.3	11.7	21.5	22.8	23.7	4	14	49
Reach B	16.0	1	1:3	440	4.0	6.5	16.8	17.7	18.5	13	39	150
Reach C	16.0	3	1:3	430	3.4	5.0	15.3	16.2	17.0	0	3	10
Reach D	15.5	5	1:3	430	3.4	5.0	15.3	16.2	17.0	0	5	15
Reach E	14.5	9	1:3	1720	2.4	3.6	12.9	14.7	15.6	0	2	6
Reach F	14.0	9	Vertical	630	2.4	3.6	12.9	14.7	15.6	0	2	4

NOTES: * Overtopping in Revere Beach Reach B is prevented from flowing into the backshore area by a dike constructed just west of Revere Beach Boulevard.

** All structure heights are based on that required for the regional plan.

*** Overtopping volumes presented in this table were developed early in the study and do not necessarily reflect all the changes which have taken place in optimizing the regional plan. From cursory analysis, these changes should only have a minor impact on overtopping volumes and revised estimates will be developed in final design.

TABLE 21

WAVE RUNUP AND OVERTOPPING - 500-YEAR DESIGN -
REGIONAL PLAN

Structure	Top Elevation** (ft. NGVD)	Ground At Toe (ft. NGVD)	Structure Slope V:H	Length Of Structure (ft)	Maximum Significant Wave (ft)	Maximum Significant Runup: Maximum Height (ft)	Maximum Significant Runup: Elevation (ft NGVD) 100-Year Event	Maximum Significant Runup: Elevation (ft NGVD) 500-Year Event	Maximum Significant Runup: Elevation (ft NGVD) SPF Event	Overtopping*** (ac-ft) 100-Year Event	Overtopping*** (ac-ft) 500-Year Event	Overtopping*** (ac-ft) SPF Event
Floodgate Structure												
Concrete	14.2	-14 to -18	Vertical	600	2.4	3.3	13.6	14.5	15.3	0	0	10
Embankment	14.2	-14 to 9	1:2	700	2.4	3.1	13.4	14.3	15.1	0	0	10
Lynn Bulkhead												
Reach A	14.2	-3	1:2	700	2.4	3.3	13.6	14.5	15.3	0	1	5
Reach B	16.0	-3	1:2	1800	3.4	4.3	14.6	15.5	16.3	0	0	0
Reach C	14.2	-3	1:2	2000	2.4	3.3	13.6	14.5	15.3	0	1	20
Reach D	14.2	-3	1:2	2550	2.4	3.3	13.6	14.5	15.3	0	0	5
Reach E	13.2	-3	1:2	1300	2.1	2.8	13.1	14.0	14.8	0	2	45
Reach F	13.2	-3	1:2	1200	2.1	2.8	13.1	14.0	14.8	0	1	30
Revere Beach												
Reach A1	16.2	14.5	Vertical	1430	10.5	2.0	12.3	13.2	14.0	0	0	0
Reach A2	19.3	12.5	Vertical	525	9.5	2.7	13.0	13.9	14.7	0	0	0
Reach B1	16.4	6	Vertical	1475	7.0	13.8	18.7	21.6	26.8	30*	120*	300*
Reach B2	20.4	1	Vertical	570	9.5	23.1	28.4	30.9	33.0	100*	205*	340*
Reach B3	16.8	3	Vertical	1515	9.0	7.5	17.0	18.5	19.5	460*	825*	1250*
Reach B4	18.0	8.5	Vertical	545	7.3	19.2	23.7	26.5	30.0	105*	240*	500*
Reach C1	16.9	13	Vertical	1355	8.6	3.9	13.7	14.7	15.9	0	0	0
Reach C2	20.4	12	Vertical	565	8.6	3.9	13.7	14.7	15.9	0	0	0
Reach C3	16.1	12	Vertical	1360	8.6	2.4	12.5	13.5	14.4		0	0
Reach C4	15.9	12	Vertical	1300	10.4	2.8	12.9	13.9	14.8	.	0	0
Reach C5	16.0	3	Concrete Steps	660	5.8	12.8	21.4	22.3	24.8	50	105	230
Reach D1	16.0	3	Concrete Steps	1500	5.8	12.8	21.4	22.3	24.8	200	395	875
Reach D2	16.0	6	Vertical	[illegible]	2.0	6.7	16.4	17.9	20.1	5	20	60
Point of Pines												
Reach A	16.5	5	1:3	230	4.3	11.7	21.5	22.8	23.7	4	14	49
Reach B	16.0	1	1:3	440	4.0	6.5	16.8	17.7	18.5	13	39	150
Reach C	16.0	3	1:3	430	3.4	5.0	15.3	16.2	17.0	0	3	10
Reach D	15.4	5	1:3	430	3.4	5.0	15.3	16.2	17.0	0	5	15
Reach E	14.5	9	1:3	1720	2.4	3.6	12.9	14.7	15.6	0	2	8
Reach F	14.0	9	Vertical	630	2.4	3.6	12.9	14.7	15.6	0	2	4

NOTES: * Overtopping in Revere Beach Reach B is prevented from flowing into the backshore area by a dike constructed just west of Revere Beach Boulevard.

** All structure heights are based on that required for the regional plan.

*** Overtopping volumes presented in this table were developed early in this study and do not necessarily reflect all the changes which have taken place in optimizing the regional plan. From cursory analysis, these should only have a minor impact on overtopping volumes and revised estimates will be developed.

TABLE 22

WAVE RUNUP AND OVERTOPPING - SPN DESIGN - REGIONAL PLAN

Structure	Top Elevation** (ft. NGVD)	Ground At Toe (ft. NGVD)	Structure Slope V:H	Length Of Structure (ft)	Maximum Significant Wave (ft)	Maximum Significant Runup: Maximum Height (ft)	Maximum Significant Runup: Elevation (ft NGVD) 100-Year Event	Maximum Significant Runup: Elevation (ft NGVD) 500-Year Event	Maximum Significant Runup: Elevation (ft NGVD) SPN Event	Overtopping*** (ac-ft) 100-Year Event	Overtopping*** (ac-ft) 500-Year Event	Overtopping*** (ac-ft) SPN Event
Floodgate Structure												
Concrete	15.0	-14 to -18	Vertical	600	2.4	3.3	13.6	14.5	15.3	0	0	0
Embankment	15.0	-14 to 9	1:2	700	2.4	3.1	13.4	14.3	15.1	0	0	0
Lynn Bulkhead												
Reach A	15	-3	1:2	700	2.4	3.3	13.6	14.5	15.3	0	0	1
Reach B	17	-3	1:2	1800	3.4	4.3	14.6	15.5	16.3	0	0	0
Reach C	15	-3	1:2	2000	2.4	3.3	13.6	14.5	15.3	0	0	1
Reach D	15	-3	1:2	2550	2.4	3.3	13.6	14.5	15.3	0	0	1
Reach E	14	-3	1:2	1300	2.1	2.8	13.1	14.0	14.8	0	0	1
Reach F	14	-3	1:2	1200	2.1	2.8	13.1	14.0	14.8	0	0	0
Revere Beach												
Reach A1	16.2	14.5	Vertical	1430	[illegible]	[illegible]	[illegible]	[illegible]	14.0	0	0	0
Reach A2	19.3	12.5	Vertical	525	9.5	2.7	13.0	13.9	14.7	0	0	0
Reach B1	16.4	6	Vertical	1475	7.0	13.8	18.7	21.6	26.8	30*	120*	300*
Reach B2	20.4	1	Vertical	570	9.5	23.1	28.4	30.9	33.0	100*	205*	340*
Reach B3	16.8	3	Vertical	1515	9.0	7.5	17.0	18.5	19.5	460*	825*	1250*
Reach B4	18.0	8.5	Vertical	545	7.3	19.2	23.7	26.5	30.0	105*	240*	500*
Reach C1	16.9	13	Vertical	1355	8.6	3.9	13.7	14.7	15.9	0	0	0
Reach C2	20.4	12	Vertical	565	8.6	3.9	13.7	14.7	15.9	0	0	0
Reach C3	16.1	12	Vertical	1360	8.6	2.4	12.5	13.5	14.4	0	0	0
Reach C4	15.9	12	Vertical	1300	10.4	2.8	12.9	13.9	14.8	0	0	0
Reach C5	16	3	Concrete Steps	660	5.8	12.8	21.4	22.3	24.8	50	105	230
Reach D1	16	3	Concrete Steps	1550	5.8	12.8	21.4	22.3	24.8	200	395	875
Reach D2	16	9	Vertical	900	2.6	6.7	16.4	17.9	20.1	5	20	50
Point of Pines												
Reach A	16.5	5	1:3	230	4.3	11.7	21.5	22.8	23.7	4	14	49
Reach B	16.0	1	1:3	440	4.0	6.5	16.8	17.7	18.5	13	39	150
Reach C	16.0	3	1:3	430	3.4	5.0	15.3	16.2	17.0	0	3	10
Reach D	15.4	5	1:3	430	2.9	5.0	15.3	16.2	17.0	0	5	15
Reach E	14.5	9	1:3	1720	2.4	3.6	12.9	14.7	15.6	0	2	8
Reach F	14.0	9	Vertical	630	2.4	3.6	12.9	14.7	15.6	0	2	4

NOTES: * Overtopping in Revere Beach Reach B is prevented from flowing into the backshore area by a dike constructed just west of Revere Beach Boulevard

** All structure heights are based on that required for the regional plan.

*** Overtopping volumes presented in this table were developed early in the study and do not necessarily reflect all the changes which have taken place in optimizing the regional plan. From cursory analysis, these changes should only have a minor impact on overtopping volumes and revised estimates will be developed in the final design.

overtopping of coastal flood protection structures are the same for both the LPP and regional plan options; with the exceptions being reaches C5 and D in the Revere Beach area where higher walls are needed in the LPP to reduce overtopping volumes. For the regional plan, allowance for storage of all overtopping within the estuary has been made.

The Shore Protection Manual was used to develop wave overtopping volumes for the entire length of Revere Beach. Overtopping relationships developed as part of the WES reports on coastal flooding at Roughans Point in September 1986, were used to estimate overtopping for the remaining areas. The reason is that no structures along Revere Beach are representative of those used in the WES model.

Using the procedure presented in Section 7.II.2 of the Shore Protection Manual, average rates of irregular wave overtopping were computed for Revere Beach for various stillwater tide levels, thus allowing for development of rating curves of tide levels versus overtopping rates. A maximum northeast wind of about 60 mph was assumed to be occurring during the period of overtopping. Tide stage hydrographs having selected maximum stillwater tide heights were then developed by appropriate adjustment of the tide hydrograph observed for 7 February 1978 which occurred during the great northeaster of 6 and 7 February 1978. Combining this information, wave overtopping hydrographs for these tidal floods were then developed for use in determining the interior flood levels. Overtopping hydrographs were developed for the existing conditions, which are assumed to remain the same with the project in place. Reaches A and most of C for the Revere Beach area did not have any significant overtopping. Although reach B of Revere Beach shows significant overtopping, an interior parkland dike is planned to prevent floodwaters from flowing inland. Heights of the interior dike are estimated at 21, 22, and 23 feet for the 100 and 500-year and SPN design events, respectively. Overtopping volumes ponding on Revere Beach Boulevard will be returned to the ocean as high tidal flood waters recede.

Reaches C5 and D will also be overtopped by waves. Floodwaters will flow in a westerly direction past the beachfront houses into the estuary where they will be temporarily stored as part of the storage capability of the regional flood plan. Under the local protection plan option, there will be insufficient storage capability in this area to safely contain floodwaters for any substantial storm; therefore, the wall in reaches C5 and D will have to be modified for the design condition.

Although overtopping was previously computed for the Point of Pines area in the 1984 DPR, it was decided to use WES overtopping relationships determined as part of the Roughans Point Study, since they not only provided a more accurate representation of the random wave climate at Point of Pines, but also were able to more closely represent the physical processes associated with overtopping of the shore front structures. The WES overtopping relationship is presented below:

$$Q = Q_o e^{C_i F'}$$

Where: Q = flowrate

C_1 = a dimensionless coefficient

Q_o = a coefficient with the same units as Q

F' = dimensionless relative freeboard term which is equal to:

$$F' = \frac{F}{(Hmo^2\ Lp)^{1/3}}$$

Where: F = freeboard, the difference between crest height of the shore front structure and the local stillwater level

Hmo = zero moment wave height

Lp = wave length

Overtopping estimates for the floodgate and Lynn bulkhead portions of the project were also developed using WES methodology. As noted, heights of the proposed structures were determined so that essentially no overtopping would occur up to the design condition (for H_s or Hmo).

Due to influence of the Point of Pines area, the total volume flowing into the estuary will be dependent upon location of the floodgate. There will be slightly more overtopping for floodgate alignments 1 and 2 than for alignments 3 to 5 since overtopping from Point of Pines reaching the estuary with the latter alignments is impeded by the amount of water that can flow over the Lynnway. Alignment alternatives are presented in the Plan Formulation Appendix.

It should be recognized that due to the large number of options and alignments considered by planners in the formulation process that refined analysis of all features was not possible at this stage. However, sufficient detail has been included to allow planners to determine project feasbility and justification. It is expected that more refined analysis of all features of the selected project (in consultation with WES) will be accomplished during detailed design investigations.

f. Design of Saugus River Tidal Floodgate

(1) Normal Tidal Currents. On 16 April 1987, current measurements were made for a mean spring tide range condition over the course of a full tide cycle (from low to high to low tide) on the downstream side of the General Edwards Bridge. During the day, concurrent measurements were made at eleven stations across the width of the bridge at one-half hour increments and the results are summarized in table 23. Between each bay of the bridge, velocities were measured at 0.2 and 0.8 percent of depth and the results were then averaged. The maximum local velocity in an individual bay for both ebb and flood conditions was 2.44 fps (1.4 knots) and the average velocity during peak discharge condition was 1.78 fps (1.1 knots). Plan views of the instantaneous currents for flood and ebb flow conditions across the width of the bridge during times of peak discharge are shown in figures 13 and 14. In general, higher velocities occurred in the middle bays of the bridge where the deeper channels are located, since bottom friction effects are reduced with increasing depth.

On closer inspection of the data, it was also noted that velocities tended to be higher on the Revere side during incoming tide and on the Lynn side during outgoing tide. This appears to be caused by the fact that a larger tidal volume is exchanged from the Pines River than the Saugus River, i.e., momentum of the tidal movement at the General Edwards Bridge is dominated by the north-south movement of the larger interchange volume of the Pines River than the east-west movement of the smaller interchange volume of the Saugus River. During the Project Engineering and Design phases, additional field data collection and a two-dimensional model will be developed to verify this condition. Background on future model studies is included in Addendum II of this appendix.

(2) Design of Gated Openings. There are several objectives which govern the selection of opening sizes for the Saugus River tidal floodgate: (a) passage of commercial and recreational navigation vessels must not be significantly impacted, (b) astronomic tide levels in the Saugus and Pines Rivers tidal wetlands should not be significantly altered, (c) present normal tidal flushing of the estuary should not

TABLE 23

<u>SUMMARY OF CURRENT MEASUREMENTS</u>
<u>GENERAL EDWARDS BRIDGE</u>
<u>16 APRIL 1987</u>
<u>TIDE RANGE -5.5 TO +5.6 TO -4.4</u>
(FPS)

Time	0830	0900	0930	1000	1030	1100	1130	1200	1230	1300	1330	1400	1430	1500	1530	1600	1630	1700	1730	1800	1830	1900	1930
Maximum Local Velocity	0.80	1.14	1.30	1.80	2.30	2.44	2.11	2.04	1.42	1.36	0.38	+.09* -.42	-.96	-1.73	-1.68	-2.04	-2.26	-2.34	-2.44	-2.20	-1.96	-1.46	-.79
Average Velocity	0.42	0.88	0.87	1.33	1.56	1.73	1.57	1.37	0.96	0.95	0.31	-0.17	-0.57	-1.21	-1.28	-1.34	-1.60	-1.66	-1.78	-1.52	-1.40	-1.08	-0.44

NOTES: Flood tide conditions occurred from 0830 to 1400 (positive velocity)

Ebb tide conditions occurred from 1400 to 1930 (negative velocity)

* Local maximum velocity of 0.09 fps occurred in one of the outside bays and -0.42 fps occurred in the center bay

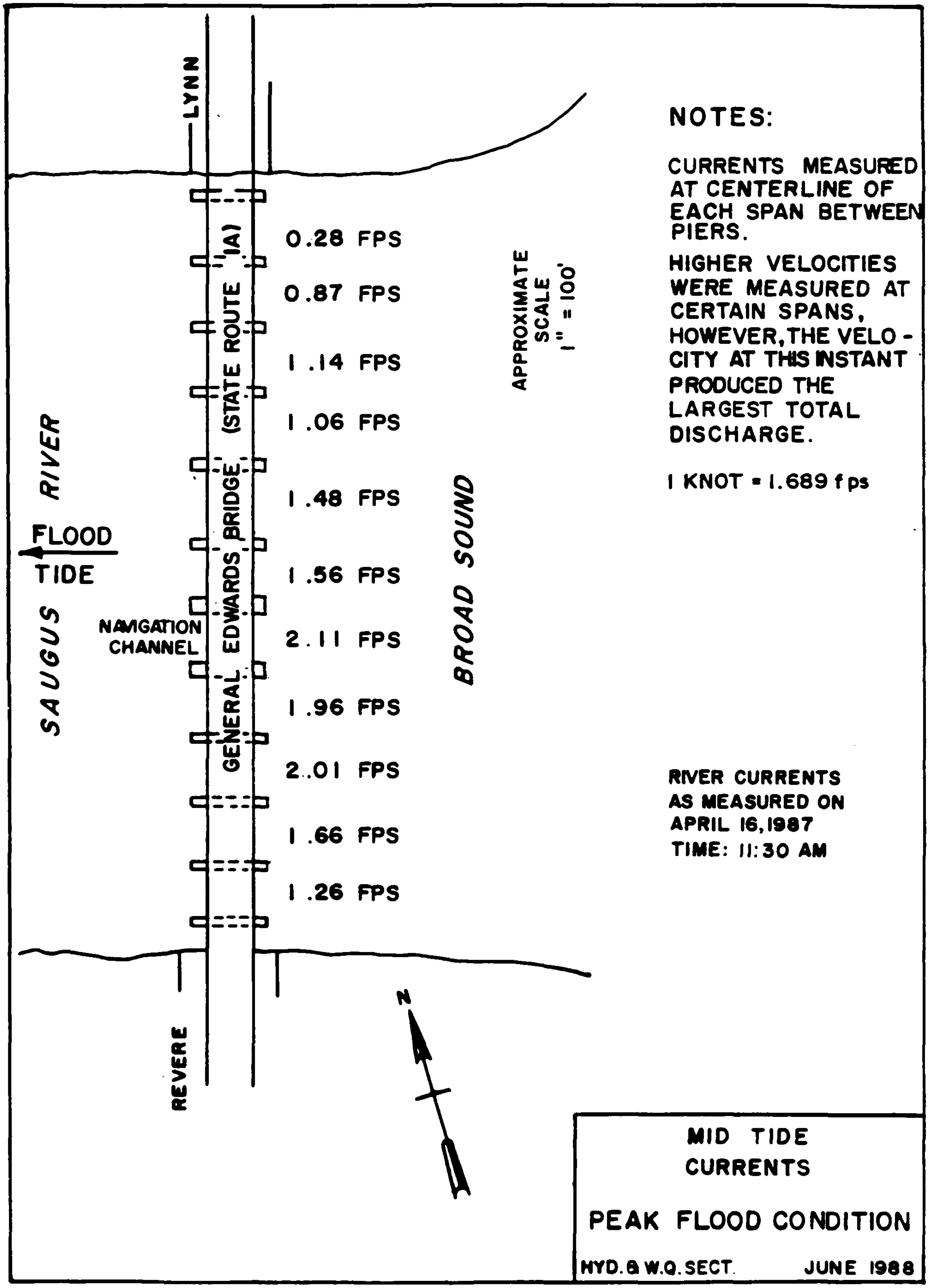

LYNN
SAUGUS RIVER
FLOOD TIDE
NAVIGATION CHANNEL
GENERAL EDWARDS BRIDGE (STATE ROUTE 1A)
0.28 FPS
0.87 FPS
1.14 FPS
1.06 FPS
1.48 FPS
1.56 FPS
2.11 FPS
1.96 FPS
2.01 FPS
1.66 FPS
1.26 FPS
BROAD SOUND
APPROXIMATE SCALE
1" = 100'
REVERE
N
NOTES:
CURRENTS MEASURED AT CENTERLINE OF EACH SPAN BETWEEN PIERS.
HIGHER VELOCITIES WERE MEASURED AT CERTAIN SPANS, HOWEVER, THE VELOCITY AT THIS INSTANT PRODUCED THE LARGEST TOTAL DISCHARGE.
1 KNOT = 1.689 fps
RIVER CURRENTS AS MEASURED ON APRIL 16, 1987
TIME: 11:30 AM
MID TIDE CURRENTS
PEAK FLOOD CONDITION
HYD. & W.Q. SECT.
JUNE 1988

FIGURE 13

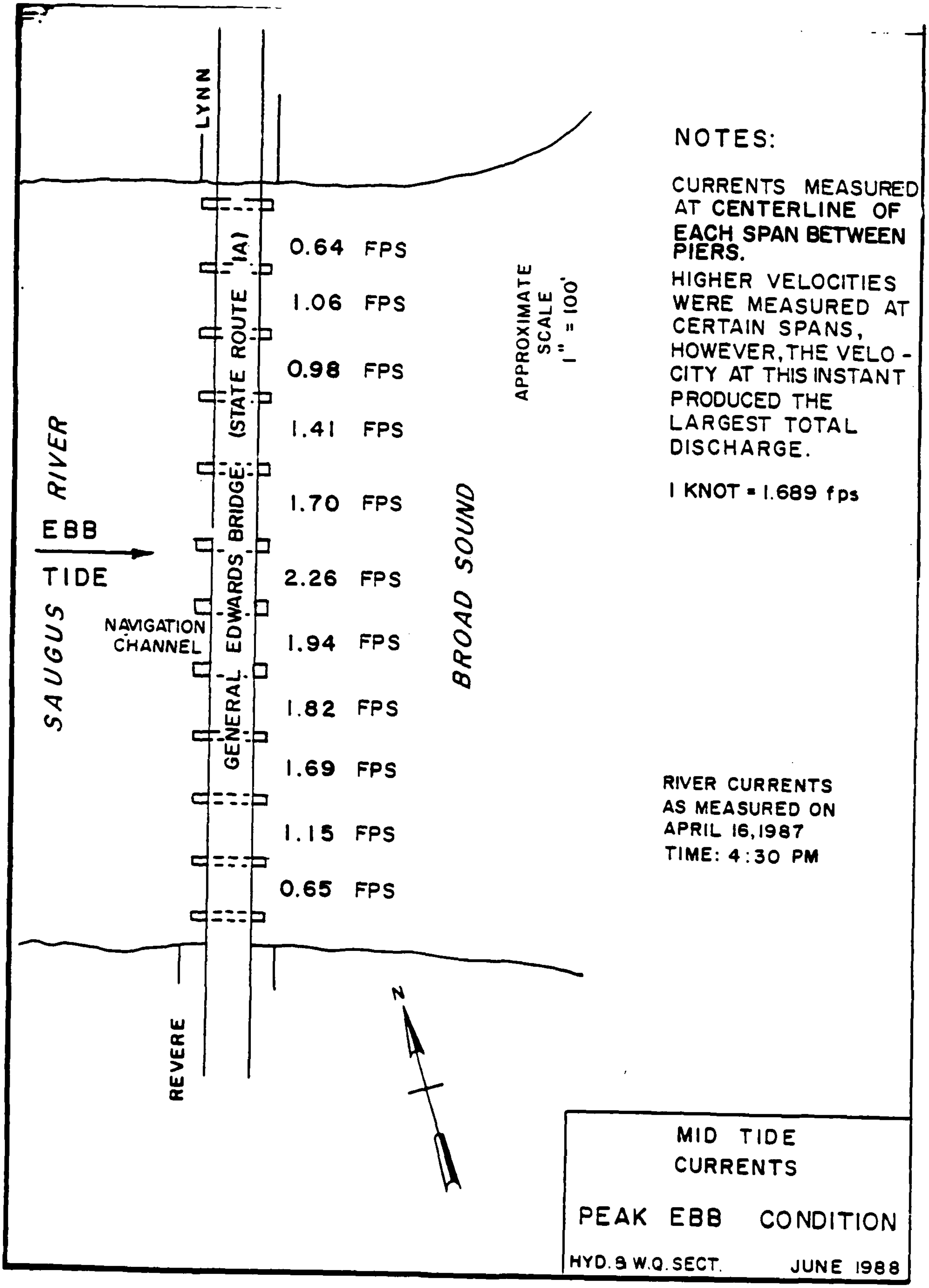

LYNN
REVERE
SAUGUS RIVER
EBB TIDE
NAVIGATION CHANNEL
GENERAL EDWARDS BRIDGE (STATE ROUTE 1A)
BROAD SOUND
0.64 FPS
1.06 FPS
0.98 FPS
1.41 FPS
1.70 FPS
2.26 FPS
1.94 FPS
1.82 FPS
1.69 FPS
1.15 FPS
0.65 FPS
APPROXIMATE SCALE 1" = 100'
N
NOTES:
CURRENTS MEASURED AT CENTERLINE OF EACH SPAN BETWEEN PIERS.
HIGHER VELOCITIES WERE MEASURED AT CERTAIN SPANS, HOWEVER, THE VELOCITY AT THIS INSTANT PRODUCED THE LARGEST TOTAL DISCHARGE.
1 KNOT = 1.689 fps
RIVER CURRENTS AS MEASURED ON APRIL 16, 1987
TIME: 4:30 PM
MID TIDE CURRENTS
PEAK EBB CONDITION
HYD. & W.Q. SECT.
JUNE 1988

FIGURE 14

be appreciably disturbed, (d) passage of fish and other biological organisms should not be measurably impeded, and (e) normal discharge of freshwater flows will not be hindered. It was determined that the navigation objective was the critical criteria. Meeting the navigation objective should generally fulfill all other objectives.

Discussion with various sources including representatives of WES, Cape Cod Canal, etc., indicates generally that if velocities are kept below about 3 knots (5.1 fps) no significant impacts to navigation will occur. A local harbor master and mariners have indicated that there are navigation problems at the General Edwards Bridge associated with both pleasure and commercial craft at the present time. Recreational boats attempting to dock at a yacht club near the bridge are hindered by fast moving currents near the mouth of the river. Commercial craft must speedup to maintain steerage near the General Edwards Bridge. Navigation problems near the bridge may be due to eddys or cross currents caused by the wide and blunt-faced bridge piers. This points out the fact that current <u>direction</u> and <u>speed</u> are important to navigation. Future design investigations will address local current speed and direction which is beyond the scope of this planning effort. Some mariners have indicated that 3 knot (5.1 fps) currents should not pose a problem to either recreational or commercial craft if added dangerous eddys and cross currents are not created. As measured, present cross sectional average velocity at the General Edwards Bridge is about one knot during mean spring tidal conditions, with maximum local velocity at the channel center being about 1.4 knots.

Velocities under proposed conditions were evaluated with the consideration that 3 knots would be the maximum velocity allowed, although ideal conditions would be not to change the maximum velocity from that which exists now. In an effort to show the sensitivity of velocity to gate opening and tidal condition, several "schemes" were analyzed (table 24).

TABLE 24

FLOODGATE OPENING SCHEMES

Schemes	Description
FC	Size openings to minimum size required to pass navigation vessels without consideration to velocity.
N1	Size openings such that maximum average velocity during mean range is about 5.1 fps (3 knots).
N2	Size openings such that maximum average velocity during mean spring range is about 5.1 fps (3 knots).
N3	Size openings such that maximum average velocity during maximum astronomic range is about 5.1 fps (3.0 knots).
N4	Size openings such that the opening below 0 foot NGVD is nearly as large as the smallest cross sectional area at the mouth of the Saugus River. This should provide similar maximum average velocities at the floodgate structure as at this smallest cross section.
EN	Size openings such that maximum average velocity during mean spring range is as close to existing conditions under the General Edwards Bridge (1.7 fps), as possible.

Gate sizing optimization is discussed in the Plan Formulation Appendix.

Velocity through gate openings was determined by hydrologic storage routing of the tide employing the latest stage-volume relationship developed from aerial mapping. Discharge characteristics of the tidal floodgate were determined by examining past WES physical model studies for hurricane barriers proposed for Narragansett Bay, Rhode Island and Wareham-Marion, Massachusetts (WES Technical Reports 2-663, 2-721, and 2-754). Specifically, the work of McNair and Grace (TR 2-663) for a similar structure at Onset, Massachusetts was used to select discharge coefficients for navigation and flushing openings. The equation used takes the form: $Q = C_s A \sqrt{2g\Delta H}$ (submerged flow) where C_s is a function of a gross head, H, and submergence, h/H, respectively. Terms used in these equations are defined as follows:

A = Cross sectional area of navigation or flushing gate opening from top of sill to surface of tailwater, ft^2

C_s = Discharge coefficient for submerged flow

g = Acceleration of gravity, ft/sec^2

h = Depth of tailwater above riprap blanket, ft

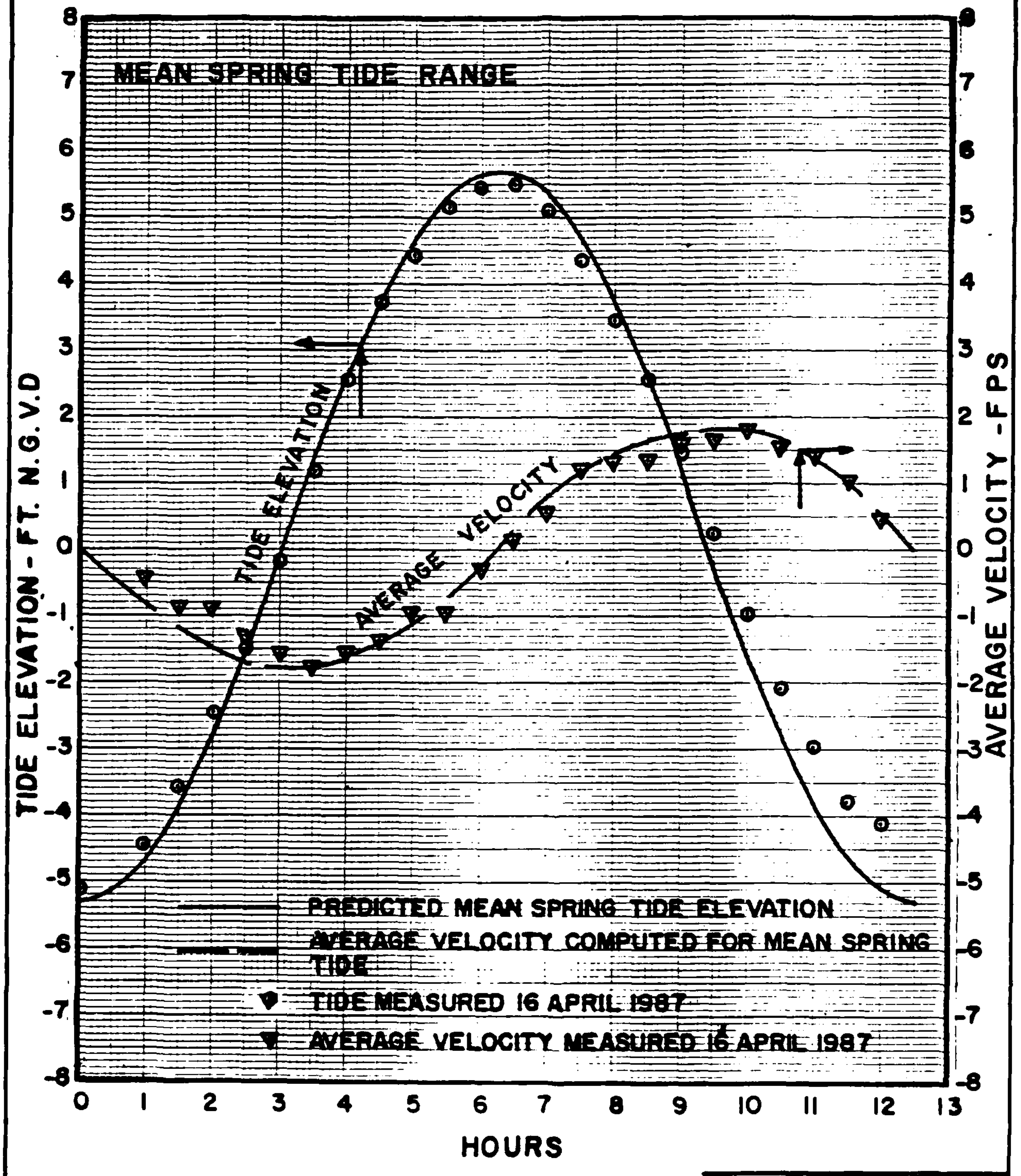
H = Gross head on riprap blanket including velocity of approach, ft

L = Average width of navigation or flushing opening based on gross head on riprap blanket, ft

Q = Total discharge, cfs

ΔH = H - h, ft

Due to the high submergence anticipated, a "C_s" of 0.65 was selected. During higher tide levels pressure flow will occur through the flushing gates. Due to the small difference in water level on either side of the structure, pressures will not be great. In this case a similar discharge equation was used except the area was limited to the gate area and a "C_s" of 0.67 was approximated. The analysis was confirmed on the existing condition by comparison with recently observed tide and velocity measurements at the General Edwards Bridge (see figure 15). The incoming tide range on 16 April 1987 was nearly a mean spring tide condition. Maximum average velocities for each scheme during various tidal conditions are shown in table 25 along with estimated cross sectional area required below mean sea level (mean sea level is the approximate elevation at which maximum flow occurs). Provided hydraulic conveyance is adequate, the sizing of individual gate openings is significantly related to mechanical, structural, operational, and economic considerations. These aspects were examined by others in consultation with hydraulic engineers to select the gate types and size shown in the main report. It should be noted that there is some flexibility in setting gate sizes, soffits (top of gated opening), and inverts (bottom of gated opening) in the final design (provided the necessary flow area is maintained) when the exact structure location and existing river cross section will be known.



NATURAL AVERAGE CURRENTS AT GENERAL EDWARDS BRIDGE

HYD. & W.Q. SECT. JULY 1988

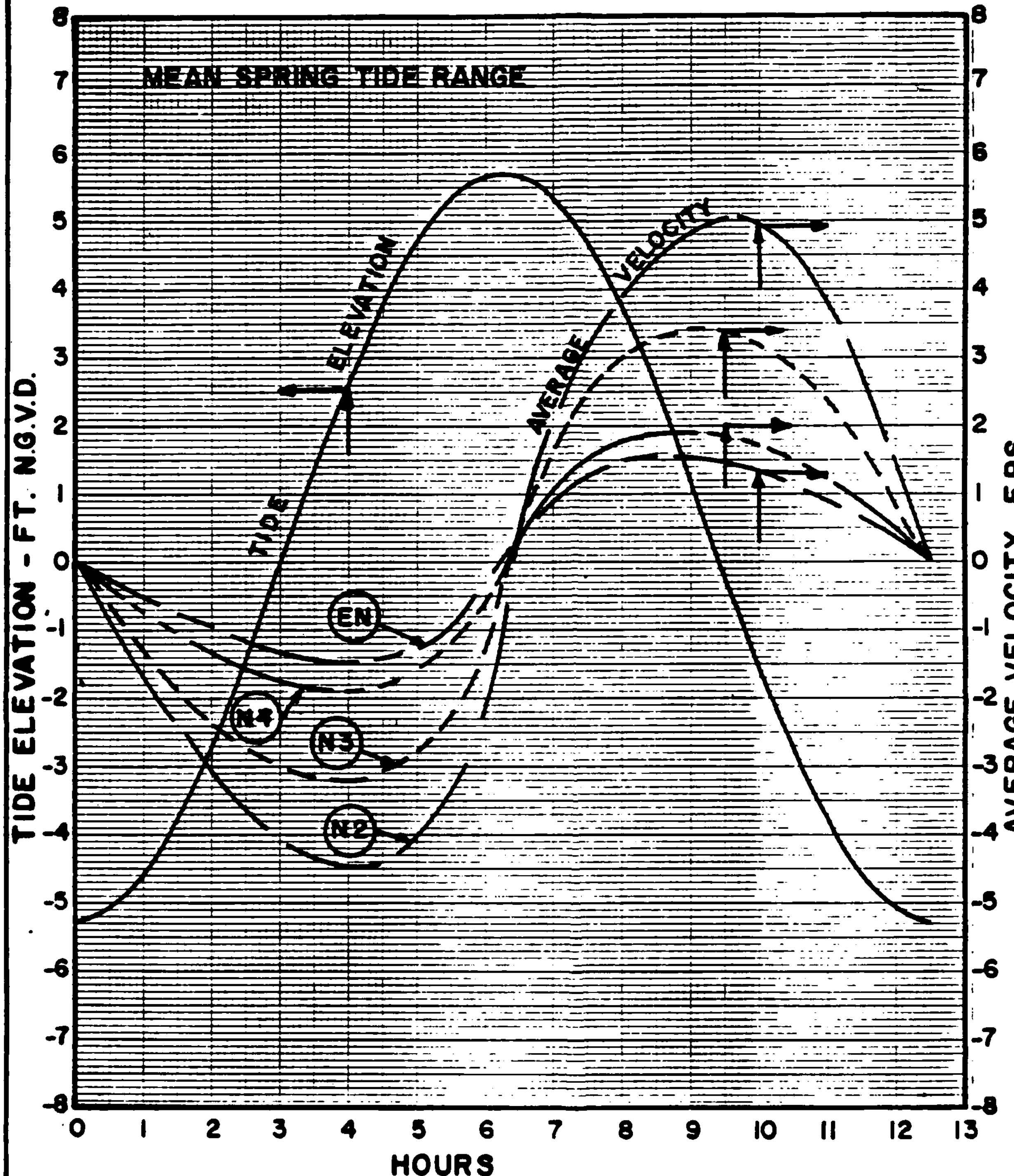
FIGURE 15

Figures 16 and 17 show velocity versus time for mean spring and maximum astronomic tidal conditions for schemes N2, N3, N4, and EN.

It should also be noted that previous WES studies and current measurements at the New Bedford, MA hurricane barrier have indicated that local point velocities in the center of openings are generally on the order of 1 to 2 fps greater than the cross sectional average velocities indicated in table 25. Local velocities can only be accurately estimated in a hydraulic model which will be accomplished in final design. The Plan Formulation Appendix includes a sensitivity analysis on selecting the gated flow area based on the 1 to 2 fps range.

An analysis of the frequency of astronomic tidal conditions and its related impacts on average velocities was completed for each scheme. Figure 2 illustrates the frequency of exceedance for astronomic tidal fall as summarized from data developed by CERC for the NOS gage in Boston. Table 26 uses this tide data as well as current data computed during the storage routing to determine how often the critical velocity of 3 knots is equalled or exceeded. By way of illustration, if scheme N2 were adopted, a maximum average velocity of 3 knots would be exceeded about 3.3 percent of the time, affecting about 291 tidal inflow or outflow periods (6.25 hour duration each) per year. The average velocity would be above 3 knots on the average of 1.0 hour for each inflow or outflow period. However, during extreme tides it could exceed 3 knots for nearly four consecutive hours. Local velocities could exceed 3 knots (5.1 fps) even more, possibly impacting about half the tides around the time of mid-tide. These higher velocities, although occurring for only a few hours at a time, could thwart safe navigation passage or cause vessels to wait until the currents subside.

If scheme N3 is adopted, maximum local velocity could exceed 3 knots during about 20 percent of the tides around the time of mid-tide, possibly imposing a navigation restriction. Scheme N4 would provide maximum local velocities generally below 3 knots and only slightly higher than that which exists at the General Edwards Bridge and fairly close to that which occurs at the narrowest cross section. Both schemes N3 and N4 also provide greater spatial flushing capability near the structure which is desirable. Scheme N4 has a good chance of not meeting strong opposition from mariners or environmentalists. However, if some degree of hindrance to navigation or biota can be accepted, schemes N2 or N3, or a compromise scheme, may be acceptable. Scheme EN keeps average velocities at present levels, as measured at



MEAN SPRING TIDAL CURRENTS - MODIFIED CONDITION AT GENERAL EDWARDS BRIDGE

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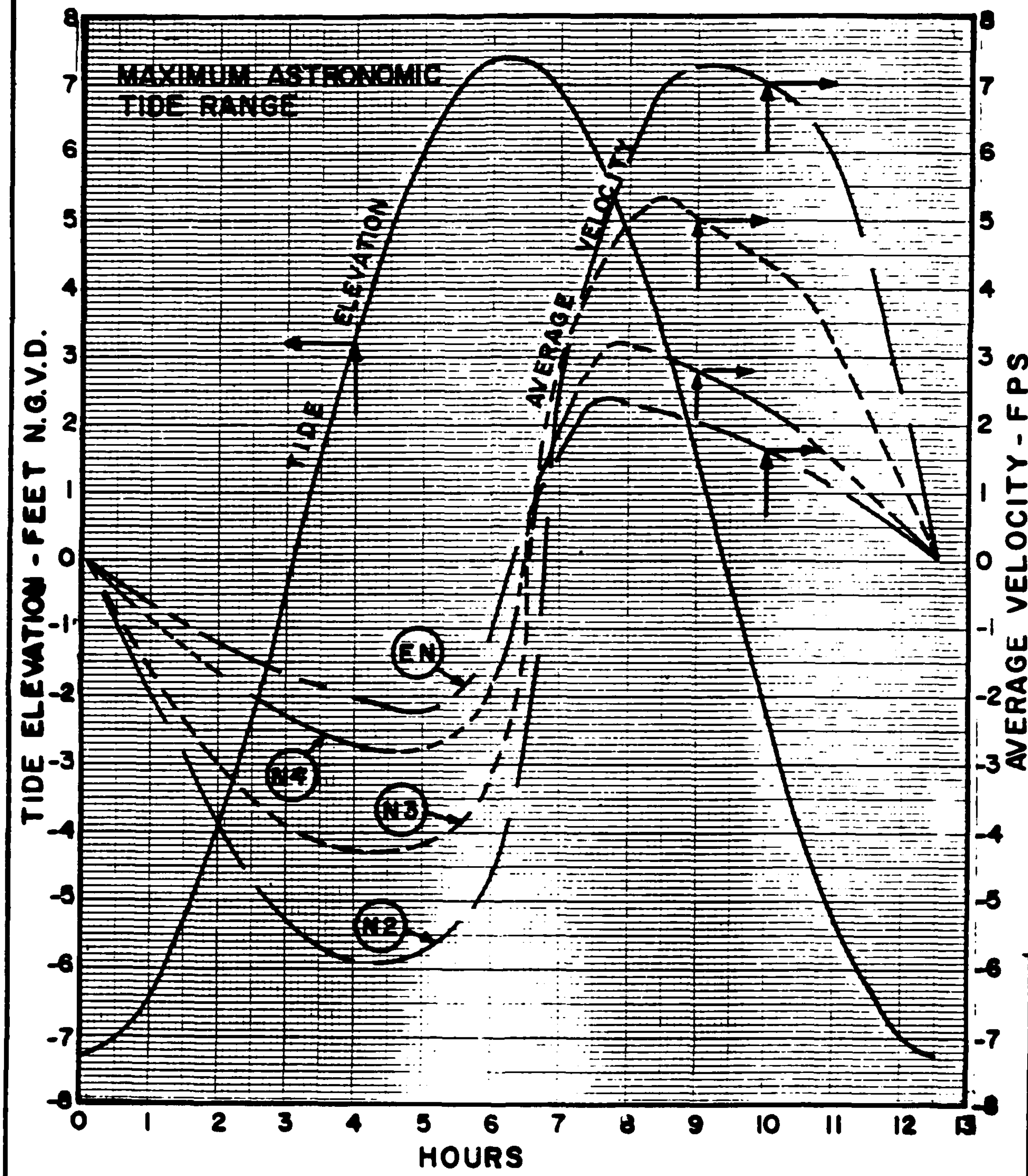
FIGURE 16



MAXIMUM ASTRONOMIC TIDAL CURRENTS - MODIFIED CONDITION AT GENERAL EDWARDS BRIDGE

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FIGURE 17

TABLE 25

MAXIMUM AVERAGE VELOCITIES
(From Hydrologic Routing Analysis)

Alternative	Flow Area Below Mean Sea Level (sq. ft.)	Maximum Average Velocity, V Max (fps)		
		Mean Range	Mean Spring Range	Maximum Astronomic Range
Existing Condition at General Edwards Bridge	12,170	-	1.7*	-
FC	1,260	9.4	10.8	14.1
N1	2,800	5.2	6.1 (2.6 hrs)**	8.8 (4.5 hrs)**
N2	3,500	4.2	5.0	7.3 (3.8 hrs)**
N3	5,200	2.9	3.4	5.3
N4 (Selected)	8,700	1.7	2.1	3.3
EN	12,170	1.3	1.6	2.4

Notes: * From gaging completed by U.S. Geological Survey.
** Number of hours during the approximate 6.25 hour period from high to low tide when 5.1 fps (3 knots) average velocity criteria will be exceeded.

TABLE 26

ESTIMATED FREQUENCY OF MODIFIED CURRENTS

Scheme	Average Annual Hours V Above 3 Knots	Percent of Time V Above 3 Knots	Maximum Consecutive Hours V Above 3 Knots	Average Consecutive Hours V Above 3 Knots
Existing	0	0	0	0
FC	7363	84.0	6.5	5.3
N1	1830	20.9	4.5	2.6
N2	291	3.3	3.8	1.0
N3	0.2	Nearly 0	0.8	0.4
N4 (Selected)	0	0	0	0
EN	0	0	0	0

Notes: V represents average cross-sectional velocity the General Edwards Bridge, but at a much larger likely expense. Schemes FC and N1 will likely be unacceptable to the public due to very frequent high currents. Change in tide levels at the Seaplane Basin would be significant with scheme FC.

A further analysis of schemes N3 and N4 was completed assuming some potential future conditions, i.e., breaching of the I-95 embankment, completion of the Saugus River navigation project, and a 1-foot rise in sea level. Results of the analysis is shown in table 27. Of the two schemes evaluated, N4 meets the 3-knot (5.1 fps) maximum velocity criteria at all times for average velocity. Local velocities would occasionally exceed the criteria. These results may add further reason to favor the N4 option keeping in mind that local velocities may be 1 to 2 fps greater than the average. The Plan Formulation Appendix includes a sensitivity analysis of possible local currents with sea level rise. Breaching of the I-95 fill is not expected to be a future condition due to opposition by Saugus residents since this action may aggravate their flooding problem.

TABLE 27

PREDICTED FUTURE VELOCITIES
(From Hydrologic Routing Analysis)

MAXIMUM AVERAGE VELOCITY
(feet/second)

Alternative	Condition A	Condition B
N3	6.3	7.4
N4 (Selected)	4.1	4.9
EN	2.9	3.6

NOTES: Condition A - Maximum astronomic tide range with break in I-95 embankment and construction of navigation project.

Condition B - Same as condition A with additional 1 foot rise in sea level. Assumes floodgate will continue to be operated only for coastal storms. Closure for high spring tides would cause maximum currents not to change much from condition A.

It is likely that a hydraulic model will be necessary in the PE&D phase to demonstrate that the smallest gate opening has been selected consistent with minimal adverse impact on navigation. In addition, the spatial impact on currents could be better defined allowing for more refined estimates of sedimentation and biological impacts.

(3) <u>Effect on Basin Tides.</u> The placement of a constriction at the mouth of the Saugus-Pines estuary requires a hydrodynamic analysis of tidal movement to accurately portray changes which result from existing interior tide stages and tidal interchange volumes. Hydraulically, the narrowness of both the Saugus and Pines Rivers, especially at highway and bridge openings, lends itself to using the selected one-dimensional model.

For the purposes of this study, ODISTM (One-Dimensional Storm Tide Model), a model developed by New England Coastal Engineers, Inc. and used by the Federal Emergency Management Agency to predict storm surges for coastal communities as part of their flood insurance studies, was employed to estimate tidal movement within the estuary. The model assumes the water velocity to be primarily undirectional and neglects the perpendicular flow direction of any of the floodwater which inundates the off channel areas, although it has the capability to characterize slow water movement in mudflats and wetlands by assuming that water goes into storage as the overbanks are flooded. By varying the downstream boundary to predict tidal movement, the link-node model uses the principles of momentum and continuity to route the tidal wave up and down the estuary. This simplified model is a useful tool for screening various alternatives in planning investigations. A more comprehensive model is planned for final project design.

Aerial photography was obtained and water surface areas delineated and measured at several different tide levels to assist in determining the surface area versus storage volume relationship behind the floodgate structure. The area-capacity relationship depicted in plate 2 is the product of this analysis.

Initially, the stage-volume relationship from the area-capacity curve, channel and marsh widths from the USGS quadrangle, and soundings collected during navigation studies were used to define the geometry of the estuary in the model. Additional adjustments were then made to the model volume by altering channel widths so that the computed tidal interchange matched fairly well to that measured with current meters on 16 April 1987 for a similar tide condition (about a mean spring tide). A single adjustable Chezy loss coefficient (C=34) was used to define impacts of constrictions, bends, and channel bed friction. In plate 11, the model discharge results for the mouth of the Saugus River are shown to match up fairly well with the actual field data for this approximate mean spring tide condition. As shown in table 28, the predicted stages and timing differences (lag from Fox Hill drawbridge on Saugus River) determined from the model, compared favorably with the data collected in the field.

Hydrodynamic analysis was completed for schemes FC, N1, N2, N3, N4, and EN (table 24); changes in high and low tide levels as compared to the existing condition were determined at several locations within the estuary. Results are shown in tables 29 and 30. As shown in the tables, water movement in the estuary is more affected by severe constrictions at low tide, than at high tide (note the values of tide changes for the FC scheme). This is understandable since at low tide the depth and flow areas are so much smaller than at the high tide condition, resulting in increased head losses and less drainage capability. Also, as shown, tide level changes are barely noticeable for schemes N2, N3, N4, and EN. In fact, the rising sea level phenomena could cause more significant future changes in tide levels (see Section 14, Rising Sea Level).

Because of the lack of topographic data in the channel upstream from the Route 107 bridge over the Pines River, exact low tide levels near the Seaplane Basin could not be accurately determined. Low tide stages determined for the station at Atlantic Lobster best indicate potential changes at the Seaplane Basin since it is the closest station unaffected by the unknowns of the Route 107 bridge and the I-95 embankment (table 30). Changes in the timing of high and low tides from existing conditions were also computed for a maximum astronomic tide range for each alternative; the results are shown on tables 31 and 32. The lag (delay in high tide level) is quite significant for schemes FC and N1, reducing to only a slight delay for schemes N3 and N4. Mean spring tide range conditions were also computed for schemes N3 and N4 and showed that for all stations the time lag reduced to less than 3 minutes and 1 minute, respectively. Neither tide level nor timing changes show major differences from existing conditions for schemes N2, N3, N4, and EN.

<u>Flushing volume changes</u> were determined for both the existing condition and schemes N3 and N4 for various tide levels. For <u>scheme N3,</u> the <u>percent reduction from existing ranged from 0.5 percent for a mean tide range to about 3 percent for maximum astronomic tide range.</u> This reduced

TABLE 28

COMPARISON OF TIDE HEIGHTS AND TIMING RELATIONSHIPS

Date	Location**	Actual Versus Dynamic Model							
		Actual		Model		Actual		Model	
		Height (ft,NGVD)	Time Lag* (hr)	Height (ft,NGVD)	Time Lag* (hr)	Height (ft,NGVD)	Time Lag* (hr)	Height (ft,NGVD)	Time Lag* (hr)
29 Jul 1984	Fox Hill Drawbridge	6.7	0	6.7	0	-6.3	0	-6.3	0
	Atlantic Lobster	6.5	0.2	6.7	0.2	-5.2	1.0	-6.4	0.2
15 Oct 1985	Fox Hill Drawbridge	7.7	0	7.7	0				
	Lincoln Avenue	7.4	0	7.7	0.1				
	Town Line Tide Gate	7.1	0.7	7.0	1.1				
14 Nov 1986	Fox Hill Drawbridge	4.7	0	4.7	0				
	Lincoln Avenue	4.6	0	4.7	0				
	Town Line Tide Gate	4.7	0.1	4.8	0.3				
16 Apr 1987	Fox Hill Drawbridge	5.6	0	5.6	0				
	Lincoln Avenue	5.6	0.3	5.6	0				
	Town Line Tide Gate	5.5	0.8	5.5	0.8				

Notes: * Time lag from Fox Hill Drawbridge (Route 107) across Saugus River

** See plate 9 for locations

TABLE 29

EFFECT ON HIGH TIDES
SAUGUS RIVER ESTUARY
(Change In High Tide Levels
From Existing Condition
For Selected Tide Conditions)
(In Feet)

	At Seaplane Basin			At Atlantic Lobster			At Lincoln Avenue		
Alternative	Mean High	Mean Spring High	Maximum Astronomic High	Mean High	Mean Spring High	Maximum Astronomic High	Mean High	Mean Spring High	Maximum Astronomic High
FC	-0.35	-0.45	-0.75	-0.30	-0.40	-1.05	-0.25	-0.40	-1.10
N1	Negligible	-0.05	-0.15	Negligible	-0.05	-0.20	Negligible	-0.05	-0.20
N2	Negligible	Negligible	-0.10	Negligible	Negligible	-0.10	Negligible	Negligible	-0.10
N3	Negligible	Negligible	-0.05	Negligible	Negligible	-0.05	Negligible	Negligible	-0.05
N4*	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible
EN	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible

*Selected

NOTES: 1. Water levels were determined using a one-dimensional dynamic routing analysis

2. The term "Negligible" refers to changes in values from existing conditions of less than 0.05 foot

TABLE 30

EFFECT ON LOW TIDES
SAUGUS RIVER ESTUARY
(Change in Low Tide Levels
from Existing Condition
for Selected Tide Conditions)
(In Feet)

Alternative	At Atlantic Lobster			At Lincoln Avenue		
	Mean Low	Mean Spring Low	Minimum Astronomic Low	Mean Low	Mean Spring Low	Minimum Astronomic Low
FC	+0.75	+1.15	+2.60	+0.70	+0.90	+1.50
N1	Negligible	+0.10	+0.20	Negligible	+0.05	+0.10
N2	Negligible	Negligible	Negligible	Negligible	Negligible	+0.05
N3	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible
N4*	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible
EN	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible

*Selected

NOTES:

1. Water levels are determined using a one-dimensional dynamic routing analysis. The use of this method in determining low tide level is extremely sensitive to channel configurations. As a result, it is felt that exact determination of alternative impacts on the Seaplane Basin are not possible at this time because channel topographical information upstream from Route 107 is spotty at best. However, low tide levels at Atlantic Lobster can be used as a good indication of what will generally happen at the Seaplane Basin.

2. The term "Negligible" refers to changes in values from existing conditions of less than 0.05 foot.

TABLE 31

TIMING DELAY IN OCCURRENCE OF HIGH TIDE
(Change From Existing Condition in Minutes)

Alternative	At Seaplane Basin	At Atlantic Lobster	At Lincoln Avenue
FC	39	63	55
N1	13	18	18
N2	9	12	16
N3	3	5	6
N4 (Selected)	0.5	0.5	1
EN	0	0	0

NOTE: This is for maximum astronomic tide range

TABLE 32

TIMING DELAY IN OCCURRENCE OF LOW TIDE
(Change From Existing Condition in minutes)

Alternative	At Atlantic Lobster	At Lincoln Avenue
FC	78	61
N1	29	11
N2	18	10
N3	8	4
N4 (Selected)	1	1
EN	0	0

NOTE: This is for maximum astronomic tide range exchange volume could potentially be offset if other constrictions, such as the abandoned I-95 embankment on the Pines River and narrow bridges on the Saugus River (i.e., the Lincoln Avenue bridge) were opened up and allowed to flush more freely and if the Saugus River navigation project was completed. The actual effect of removing constrictions is not easily addressed. A detailed study should be conducted to examine these options before any such decisions are made. For scheme N4, there was less than 0.1 percent change in flushing volume for all conditions.

As a result of velocity considerations, scheme N4 was selected by planners for the proposed plan because it is the most promising from a navigation perspective without making the floodgate structure cost excessive and does not significantly alter tide levels or flushing in the estuary. Detailed design modelling will be conducted to refine this conclusion.

(4) Sediment Movement. Analysis of the hydrology of the Saugus and Pines Rivers estuary and review of suspended solids data collected in 1986 and sediment data collected as part of the navigation study within the estuary indicate that construction of a floodgate structure across the mouth of the Saugus River will have only minimal effect on the sediment movement which now exists within the tidal area.

The upstream freshwater portion of the Saugus and Pines Rivers contributes only minor sediment load under average flow conditions because of the hydrologically slow basin characteristics (short stream lengths interconnected by large wetland areas). Average slope of the main stem of the Saugus River (the major contributor to sediment) is about 6 feet/mile and about 10 percent of the freshwater portion of the basin is comprised of small lakes and wetlands. Most sediment entrained in the stream-like portions of the river would have settled out in the freshwater wetland areas. It is likely that if any sediment movement takes place from the upper basin it will only occur during major freshwater runoff events. Sediment moving downstream from the freshwater portion during these events will be deposited rather quickly in the upper end of the tidal estuary since the river channel widens dramatically at the upper end of the estuary.

After deposition, sediment transport in the estuary is controlled by tidal movement rather than freshwater flow since freshwater flow is such a minor component of the total water movement. Table 33 shows the ratio of peak freshwater flow rates for various runoff events to the average tidal interchange flow rate for a mean tide range. As noted, even for a 100-year runoff event, the freshwater flow rate is less than one-half of the average tidal flow.

TABLE 33

SALTWATER-FRESHWATER FLOW RATES
(Estimated for Saugus River at General Edwards Bridge)

Freshwater Peak Flow Rate (cfs)	Chance of Occurrence (percent)	Average Tidal Flow Rate for Mean Tide Range (cfs)	Saltwater to Freshwater Flow Ratio
850	50 (2-yr event)	9,400	11.1
1,900	10 (10-yr event)	9,400	4.9
3,300	2 (50-yr event)	9,400	2.8
4,000	1 (100-yr event)	9,400	2.4

NOTES: Peak flow rates were obtained by statistical analysis of the Aberjona River USGS gate at Winchester, MA for the upper watershed and adopted lower watershed discharges, assuming the two peaks coincident. This flow rate does not necessarily last for 6 hours (1/2 of a tide cycle).

Average tidal flow rate is obtained by taking the tidal volume exchanged in a complete tide cycle and dividing by 6 hours.

Very little change is expected to take place in the sediment movement capability of the Saugus estuary as a result of this project. Existing and proposed average velocities (floodgate schemes N3 and N4) estimated by the hydrodynamic model for peak discharge conditions for a mean spring tide range are displayed at various locations along the Saugus and Pines Rivers in table 34. Minimal changes in velocities are anticipated throughout the estuary with the floodgate structure in place, with the exception of the area within a few hundred feet of the floodgate. Highest velocities for both proposed and existing conditions occur near the mouth of the river with other high velocities occurring at bridge crossings. Velocities in a majority of the estuary are low, less than 0.50 fps, and as a result (even under proposed conditions) currents are only capable of moving extremely fine grain or flocculated materials. Even increasing the velocities by about one-third for both existing and proposed conditions, the approximate increase if a 100-year runoff event occurs concurrent with a mean spring tide range condition, would cause only a slight increase in the sediment carrying capability of the flow. Any increase in sediment load capability may not even be measurable when considering the large quiescent areas that will remain, such as: (a) at the inside of the numerous bends, (b) along the shallow mudflat areas, and (c) upstream from bridge crossing constrictions.

Sediment data has also been collected as part of NED's navigation investigations in the Saugus and Pines Rivers. Data collected from these studies are included in tables 35A through 35E and location of the samples collected are shown in plate 12. In addition, bottom samples were collected near the north side of the Saugus River terminus as part of this flood control study and show very little fine material. The data collected confirms that the highest velocities occur near the mouth of the Saugus River since there is very little fine silt material build-up and bottom material generally consists of fine to medium sands. Areas in the overbanks generally show a build-up of fine silt as would be expected since currents in these areas are significantly less than those in the main channel. The data also shows the impact that street runoff and combined sewer overflow have on the sediment quality just upstream from Route 107 on the Saugus River. This is the area where the Lynn Water and Sewer Commission's Summer Street combined sewer overflow is located. The sediments both upstream and downstream from this area are extremely fine-grained and have a high organic content. Recently additional sediment samples were taken for environmental assessment purposes along Lynn Harbor and at the mouth of the Saugus River. This information is presented

TABLE 34

MAXIMUM AVERAGE VELOCITIES
(During Peak Discharge Spring Tide Condition - FPS)

Location	Existing Condition	Proposed Condition N3	Proposed Condition N4 (Selected)
Saugus River			
500 Feet Upstream from General Edwards Bridge	0.50	0.54	0.53
3,000 Feet Upstream from General Edwards Bridge	0.53	0.40	0.38
Route 107 Bridge	1.35	1.35	1.36
2,500 Feet Upstream of Route 107	0.18	0.27	0.27
1,000 Feet Downstream of Lincoln Avenue Bridge	1.27	1.24	1.20
Lincoln Avenue Bridge	2.71	2.68	2.69
2,500 Feet Downstream of Saugus Iron Works	0.07	0.06	0.07
Pines River			
4,000 Feet Upstream of General Edwards Bridge	0.51	0.49	0.49
2,000 Feet Downstream of Route 107 Bridge	0.34	0.34	0.34
Route 107 Bridge	3.05	3.06	3.03
500 Feet Upstream of Route 107 Bridge	0.12	0.12	0.12
1,500 Feet Upstream of I-95 Embankment	0.38	0.41	0.36

NOTE: Velocities are extracted from Hydrodynamic Model

TABLE 35A

PINES RIVER
REVERE, MASSACHUSETTS
BULK SEDIMENT ANALYSIS

STATION	SEPTEMBER 1982			DECEMBER 1982		APRIL 1984	
	A	B	C	D	E	F	G
Depth (ft)	Surface	Surface	Surface	Surface	Surface	Surface	Surface
Soil Description	coarse to fine sand	gravelly coarse fine sand	gravelly med. to fine sand	organic fine to fine sand	organic fine sandy silt	gravelly coarse silty sand	coarse to fine sand
Med. Grain Size (mm)	0.3200	1.3000	3.2000	0.0720	0.0940	0.3900	0.5200
% Fines	2.0	2.0	2.0	52.0	42.0	<1.0	<1.0
Oil and Grease (ppm)	-	-	-	320	657	16	11
Lead (ppm)				111	74	<19	<19
Zinc (ppm)				84	76	26	25
Cadmium (ppm)				4	<2	<2	<2
Chromium (ppm)				85	75	<19	<19
Copper (ppm)				17	27	<4	<4
PCB (ppb)				<10	<10	46	30

TABLE 35B

SAUGUS RIVER
LYNN, MASSACHUSETTS
SEDIMENT ANALYSIS

September 1982

Station	A			B			C		
Depth (ft)	0.0-1.9	0.00-0.25	1.2-1.5	0.0-1.8	0.0-0.25	1.55-1.8	0.0-1.9	0.0-0.25	1.65-1.9
Soil Description	Organic Fine Sandy Silt			Organic Fine Sandy Clayey Silt			Organic Fine Sandy Clay Silt		
Med. Grain Size (mm)	0.021	-	-	0.032	-	-	0.0190	-	-
% Fines	77	-	-	66	-	-	82	-	-
Oil & Grease (ppm)	-	1200	-	-	2200	-	-	2200	-
Lead (ppm)	-	172	262	-	320	364	-	138	103
Zinc (ppm)	-	206	264	-	336	445	-	227	171
Cadmium (ppm)	-	5	9	-	10	14	-	8	12
Chromium (ppm)	-	148	292	-	224	384	-	174	258
Copper (ppm)	-	77	124	-	126	184	-	84	77
PCB (ppb)	-	-	-	-	-	-			

TABLE 35B (cont.)

SAUGUS RIVER
LYNN, MASSACHUSETTS
SEDIMENT ANALYSIS

September 1982

Station	D			E		
Depth (ft)	0.0-2.1	0.0-0.25	1.65-1.9	0.0-1.3	0.0-0.25	1.3-1.35
Soil Description	Organic Fine Sandy Clay Silt			Silt Fine Sand		
Med. Grain Size (mm) % Fines	0.0700 51	-	-	0.1000 44	-	-
Oil & Grease (ppm)	-	1100	-	-	<120	-
Lead (ppm)	-	113	16	-	10	16
Zinc (ppm)	-	151	104	-	58	28
Cadmium (ppm)	-	8	4	-	<3	<3
Chromium (ppm)	-	133	41	-	<5	7
Copper (ppm)	-	<20	<20	-	<20	<20
PCB (ppb)	-	<10	-	-	<10	-

TABLE 35C

SAUGUS RIVER
LYNN, MASSACHUSETTS
BULK SEDIMENT ANALYSIS

April 1984

Station	F	G			H			I		
Depth (ft)	Surface	0.0-2.3	0.0-0.25	2.3-2.6	0.0-1.1	0.0-0.25	1.2-1.5	0.0-1.0	0.0-0.25	1.3-1.6
Soil Description	Silty Fine Sand	Organic Sandy Silt			Organic Sandy Silt			Med. to Fine Sand		
Med. Grain Size (mm)	0.0800	0.0350	-	-	0.0400	-	-	0.04200	-	-
% Fines	48	67	-	-	55	-	-	3	-	-
Lead (ppm)	50	-	103	37	-	<19	<19	-	<18	<18
Zinc (ppm)	126	-	180	141	-	58	64	-	40	76
Cadmium (ppm)	<2	-	<2	<2	-	<2	<2	-	<2	<2
Chromium (ppm)	118	-	197	162	-	62	42	-	32	75
Copper (ppm)	37	-	53	46	-	<21	23	-	<20	<21
PCB (ppb)	108	-	369	-	-	53	-	-	58	-

TABLE 35D

SAUGUS RIVER
SAUGUS AND LYNN, MASSACHUSETTS
SEDIMENT ANALYSIS

November 1984

Station	J	K	L	M	N	O
Depth	Surface	Surface	Surface	Surface	Surface	Surface
Soil Description	organic sandy silt	organic sandy silt coarse to fine sand	organic sandy silt	organic sandy silt	organic silty med. to fine sand	organic silty med. to fine sand
Med. Grain (mm)	0.0210	0.0810	0.0320	0.0250	0.1450	0.1150
% Fines	92	47	76	85	34	33
Specific Gravity	2.60	2.66	2.61	2.63	2.63	2.65

TABLE 35E

SAUGUS RIVER
SAUGUS AND LYNN, MASSACHUSETTS
SEDIMENT ANALYSIS

February 1986

Station	P	Q	R	S
Depth (ft)	Surface	Surface	Surface	Surface
Soil Description	sand	silty sand	sand with trace of silt	sand with trace of silt
Med. Grain Size (mm)	0.5200	0.0950	0.0800	0.1300
% Fines	0.5	14.5	8.0	7.5
Specific Gravity	2.70	2.68	2.68	2.71

in the Environmental Appendix and shows no major changes from previous data.

Overall, the sediment data collected shows that velocities in general are low throughout the estuary (as is consistent with the hydrodynamic results) since bottom material sampled range from sandy silt to coarse sand with little or no gravel or cobble type material. Higher velocities would have resulted in larger particle sizes along the channel surface. Under normal conditions, with the proposed floodgate open, only the area within a few hundred feet of the project will be impacted by the restriction. A riprap apron will be necessary near the structure to prevent any undesired scour. Additional sediment build-up may occur near the quiescent areas upstream and downstream from the floodgate structure. Areas where additional sediment may build-up and where increased erosion may take place are shown in plate 13. A model will be completed in the Project Engineering and Design phases to verify this assumption (see Addendum II). Also, areas of potential increased scour near the structures will be identified and appropriate corrective measures designed.

With the floodgate closed, which would happen only on infrequent occasions, sediment movement from the upstream basin and from storm drains may temporarily change from the existing condition. For example, post-project sediment movement will show a temporary decrease if a comparison is made to preproject conditions when peak freshwater discharge is occurring near mid-tide. However, if post-project sediment movement is compared to preproject conditions when it is slack low or slack high tide, there would be very little change in sediment movement noticeable. In any event, since bed material transported is so fine, any change in sediment movement should be minor. When comparing preproject to post-project gate closed conditions, after a short gate closure (generally 1 to 2 hours), subsequent tidal cycles would reentrain the sediment as under the existing conditions.

12. DESIGN FLOODS

a. Design Ocean Storm Criteria. See section on Tidal Hydraulics.

b. Interior Runoff Criteria. Establishing interior drainage criteria for a regional flood control project is not a definitive hydrologic process. The assessment of interior drainage needs cannot be entirely analytical, but must be a combination of both quantitative and subjective analysis. The sizing of interior drainage facilities, an indicator of cost, must be weighed against the interior flood risk based on both probability, or frequency, and magnitude of potential flood damages. For example, street drains serving small drainage areas where flood potential is minor and of a nuisance category, are usually sized using a relatively low (frequency) storm runoff criteria, i.e., 2 to 10-year storm frequency runoff. However, for systems with greater flood damage potential, possibly involving loss of life or extensive property damage, a more severe criteria is generally employed, i.e., 50 to 100-year storm frequency runoff criteria. Some hydrologic factors considered in establishing interior drainage requirements are: (a) the frequency and duration of storm tides preempting gravity drainage, (b) interior runoff potential, (c) likely coincidence of interior runoff with storm tide (discussed in paragraph 9), and (d) relative degree of primary protection sought by regional project, i.e., the higher the optimum design flood for the overall project the higher the interior drainage criteria. Based on the above considerations, interior drainage design criteria adopted for the subject regional plan, was a 10 percent (10-year) interior design runoff as criteria for a 1 percent (100-year) primary project level of design against tidal flooding, a 2 percent (50-year) interior design for a 0.2 percent (500-year) primary design, and a 1 percent (100-year) interior design for an SPN primary level of design.

c. Interior Storage Capacity. A necessary component of any Regional Saugus River Floodgate Plan would be the preservation of adequate storage capacity in the tidal estuary for safely storing design interior runoff volume, and residual tidal overtopping. As set forth in ER 1105-2-20, "Project Purpose Planning Guidance," local assurances would be needed to protect the Federal investment and assure the achievement of expected project benefits throughout project life. Accordingly, ample estuary lands for interior storage capacity would need to be preserved. Total design interior storage required for different levels of project design are listed in table 36. Volumes are based on a design gate closure period of 4.5 hours for a 100-year design, 5.5 hours for a 500-year design, and 6 hours for the SPN design. At the time this analysis was conducted the Point of Pines project area was not a part of the Regional Plan. Due to the uncertainties involved with the Point of Pines project, wave overtopping volumes were computed assuming existing conditions at Point of Pines. Since the Point of Pines project is now a component of the Regional Plan, these overtopping volumes will be somewhat reduced; however, due to the relatively small size of the Point of Pines area, reductions in overtopping volumes will not be significant. All overtopping volumes will be reviewed and updated in design studies.

TABLE 36

INTERIOR STORAGE CAPACITY REQUIREMENTS

	1% (100-yr) Design Tide	0.2% (500-yr) Design Tide	SPN* Design Tide
Saugus River Inflow (26 sq. mi.)	340 Ac-Ft (10%Q=900 cfs)	600 Ac-Ft (2%Q=1,300)	750 Ac-Ft (1%Q=1,500)
Local Runoff (21 sq. mi.)	1,650 Ac-Ft (2.2" R.O.)	3,100 Ac-Ft (3.1" R.O.)	3,500 Ac-Ft (3.5" R.O.)
Wave Overtopping	325 Ac-Ft	690 Ac-Ft	1,560 Ac-Ft
TOTAL	2,300 Ac-Ft	4,400 Ac-Ft	5,800 Ac-Ft

* Selected

There presently exists sufficient estuary area for the safe storage of interior runoff during periods of floodgate closure, eliminating the need for a costly pumping station. However, assurances of the functional integrity of such storage is critically important as stated in EC 1110-2-247, "Hydrologic Analysis of Interior Areas," dated January 1984. It was determined, using the developed area-capacity curves for the Saugus-Pines River estuary, that preserving all lands, at and below +7.0 feet NGVD in the estuary, would assure 5,400 acre-feet of storage capacity between +2.0 and +8.0 feet NGVD, assuming a vertical rise between elevations 7.0 and 8.0. Hydrologically, it is recommended that 5,400 acre-feet of storage capacity be adopted as the design interior storage capacity requirement. Local assurances would need to stipulate that no filling (loss of storage) be permitted below elevation 7.0 feet NGVD without compensating storage between elevations 5.0 and 7.0 feet NGVD. As part of project design, detailed mapping (refer to paragraph 5b) should be prepared defining the area required for hydrologic purposes. Executive Order 11988 requires the Corps to avoid inducing development in the base flood plain if there is an alternative (ER 1105-2-20). In developing areas the local sponsor is required, to the extent legally empowered, to adopt flood plain management programs to ensure wise use of the flood plains in, as well as adjacent to, the project area (ER 1105-2-20). To limit potential flood-prone development and adverse impacts on project operation, local assurances should stipulate that any new developments bordering directly on the estuary above elevation +7.0 feet NGVD require minimum lot elevations not less than +8.5 feet NGVD with first floor grades not less than +9.0 feet NGVD.

d. Residual Interior Drainage Needs. In addition to, but separate from the Saugus and Pines Rivers estuary, there are localized low areas in Revere and Saugus whose drainage is presently dependent on localized ponding and flap-gated drains to the estuary, i.e., Revere zones 2B, 4A, and 4C, and Saugus zone 2. The tidal floodgate project cannot be operated to provide continuous gravity drainage for these low areas and some residual local drainage problems, in the absence of tidal flooding, may persist. Local assurances will need to stipulate that all existing flap-gate structures to the estuary be maintained in good operating condition. Further, all proposed new developments must be reviewed with regards to their potential impact on existing drainage conditions.

All needed improvements for the handling of interior storm drainage, in the absence of tidal flooding, will be a non-Federal responsibility in accordance with ER 1165-2-21, "Flood Damage Reduction Measures in Urban Areas," dated October 1980.

e. Design Height of Protection Criteria. See section on Tidal Hydraulics.

f. Freeboard. See section on Tidal Hydraulics.

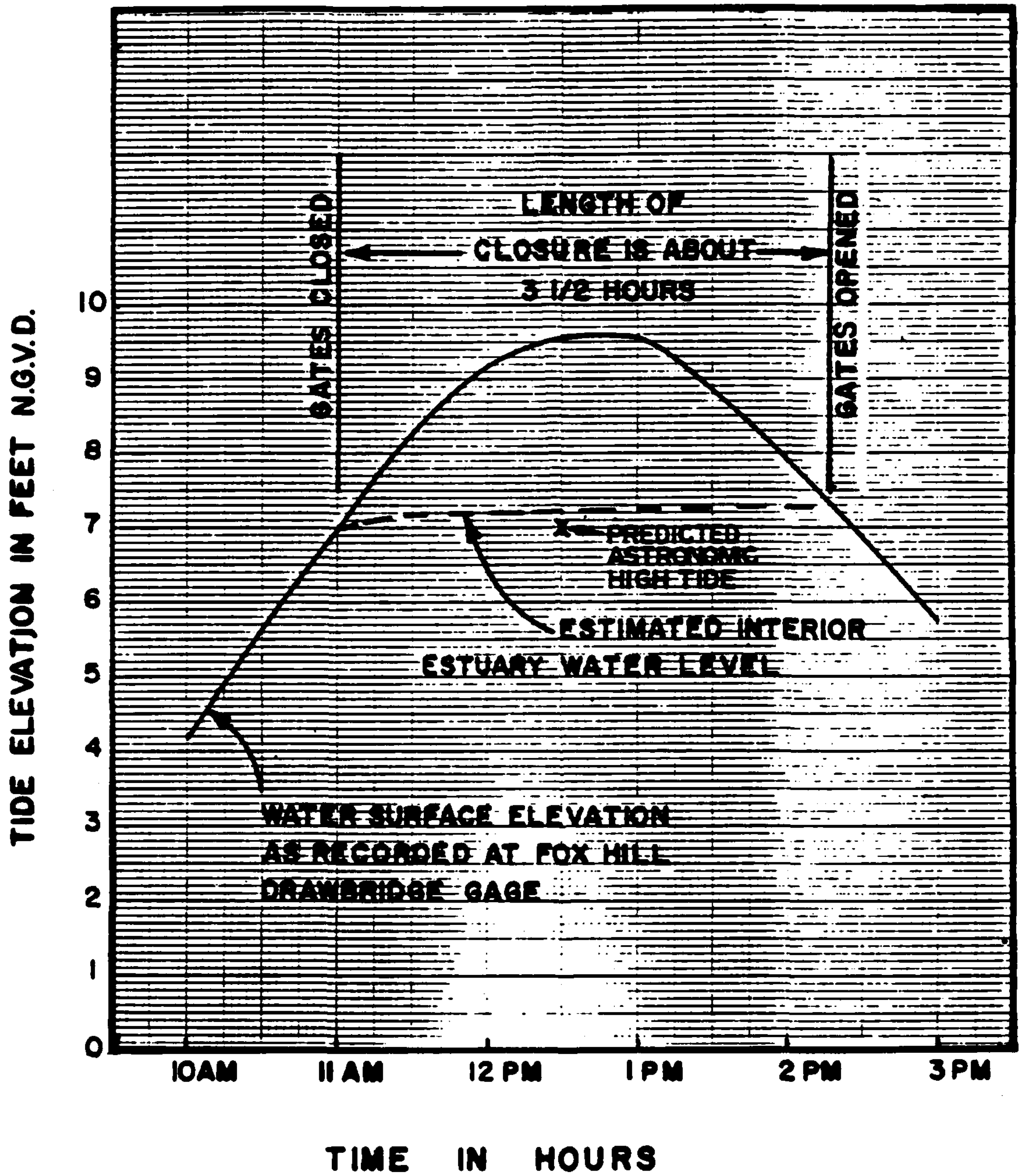
13. PROJECT OPERATION

a. General. A Saugus River Floodgate project would, under normal conditions, have gates in an open position with resulting minimal effect on the present tidal regime of the estuary. Under storm tide conditions, the gate would be closed in an effort to prevent tide levels in the estuary from exceeding about +8.0 feet NGVD. In so doing, with allowance for interior runoff and wind generated interior wave action, it would be expected that the regulated level, on average, would be more nearly +7.0 feet NGVD with infrequent rises to nearly +8.0 feet NGVD. This is not to infer that the gates will be closed every forecasted tide of +7.0 feet. Since the duration of gate closure and potential for interior runoff storage is proportional to the height of storm tide, time of closure would be highly dependent on forecasted tide and weather (rainfall) conditions. For example, if a tide of 7.5 feet is forecasted (5 to 6 times per year on average) and there had been little rain, then the gate would likely not be operated, with a resulting unmodified tide of about 7.5 feet. By comparison, if a tide of

+9.0 feet (10-year event) were forecasted, with moderate rain, then the gate would likely be closed between +6.0 and +7.0 feet NGVD. Examples of anticipated operations for three different storm events are shown in figures 18, 18A, and 19. Figure 18 depicts what would likely happen during what could be considered a 10 to 20-year tidal event which occurred on 2 January 1987. There was one peak storm tide which went up to about 9.6 feet NGVD. Figure 18A shows a typical annual event. Although start of significant damage is about elevation 8.0 feet NGVD (70 percent chance of being equalled or exceeded annually), some of the 2 to 3 closures per year would be false alarms such as this, with the closure decision being made in anticipation of potential significant flooding.

The coastal storm which occurred during 6-7 February 1978 produced two extremely high tides which were measured at the Boston NOS gage. This gage showed that the storm tide which occurred remained above 8 feet NGVD for approximately 2-1/2 hours on the first high tide dropping to less than 5 feet NGVD for about 7 hours and then rising to above 8 feet NGVD for about 3-1/2 hours on the second high tide. Hypothetical floodgate operation is shown on figure 19. This is based on assumed start of damage at 8 feet ± NGVD, precipitation during the storm being all snow (temperature less than 32 degrees Fahrenheit), and adequate prior warning given to navigation interests regarding gate closure. It can be seen that the gates would have been closed four times for a total of about 20 hours over a 3-day period. Since the storm surge lingered on for several days, closures would have occurred at four high tides between 6 and 8 February. This was the worst storm of recent history (about a 100-year event).

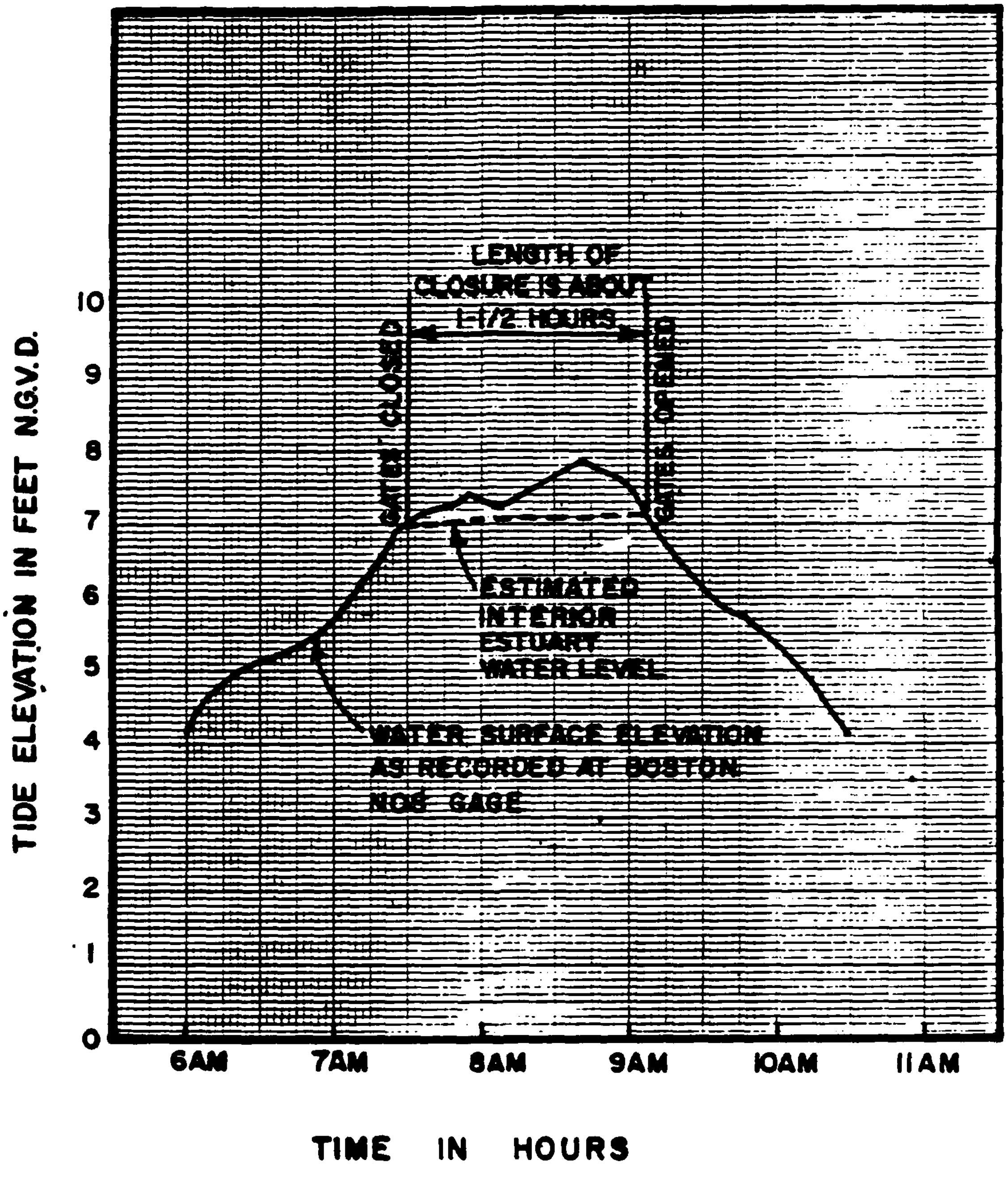
b. Design Storm Operation. In contrast to the above operating procedure, the operating criteria for a forecasted hurricane or severe northeaster with potential ocean tides in excess of 10 feet NGVD, severe winds, and torrential rains (something in the one percent chance range or rarer), the project would be closed conservatively early. For SPN design operation and design interior rainfall runoff, the project would be closed at about a +2.0 feet tide, allowing for design interior storage and remain closed up to 4 to 6 hours. A preliminary schedule of gate closure elevation versus forecasted storm tide and probable resulting maximum interior storage level is graphically illustrated on plate 7. This plate is presented as a preliminary analysis to demonstrate project capability and is not meant to infer any final gate operation schedule. Actual gate closure elevations and schedule would be developed as part of the Operation and Maintenance Manual and could be modified slightly based on actual operating experience.



ESTIMATED FLOOD GATE OPERATION FOR TIDAL SURGE OF JANUARY 2, 1987

HYD.& W.Q.SECT. JUNE 1988

FIGURE 18

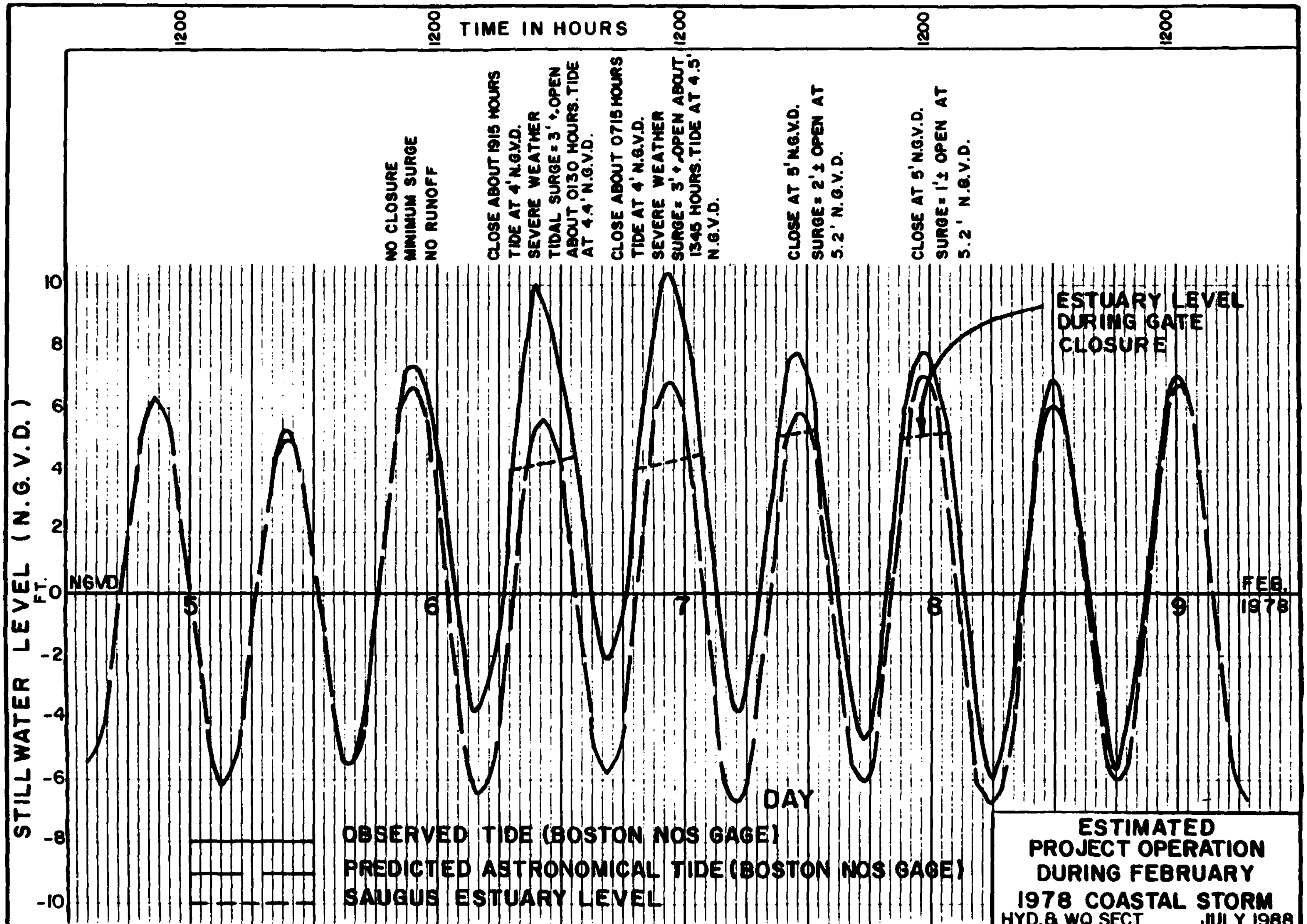


PREDICTED HIGH TIDE =
+5.5 FT, NGVD AT 8:09 AM

ESTIMATED FLOOD GATE OPERATION FOR TIDAL SURGE OF OCTOBER 22, 1988

HYD. & W.Q. SECT. NOV. 1988

FIGURE 18A



TIME IN HOURS
1200
1200
1200
1200
1200
NO CLOSURE
MINIMUM SURGE
NO RUNOFF
CLOSE ABOUT 1915 HOURS
TIDE AT 4' N.G.V.D.
SEVERE WEATHER
TIDAL SURGE = 3' +, OPEN
ABOUT 0130 HOURS. TIDE
AT 4.4' N.G.V.D.
CLOSE ABOUT 0715 HOURS
TIDE AT 4' N.G.V.D.
SEVERE WEATHER
SURGE = 3' +, OPEN ABOUT
1345 HOURS. TIDE AT 4.5'
N.G.V.D.
CLOSE AT 5' N.G.V.D.
SURGE = 2' ± OPEN AT
5.2' N.G.V.D.
CLOSE AT 5' N.G.V.D.
SURGE = 1' ± OPEN AT
5.2' N.G.V.D.
ESTUARY LEVEL
DURING GATE
CLOSURE
STILLWATER LEVEL (N.G.V.D.)
FT.
10
8
6
4
2
0
-2
-4
-6
-8
-10
NGVD
5
6
7
8
9
FEB.
1978
DAY
OBSERVED TIDE (BOSTON NOS GAGE)
PREDICTED ASTRONOMICAL TIDE (BOSTON NOS GAGE)
SAUGUS ESTUARY LEVEL
ESTIMATED
PROJECT OPERATION
DURING FEBRUARY
1978 COASTAL STORM
HYD. & WQ. SECT.
JULY 1988

FIGURE 19

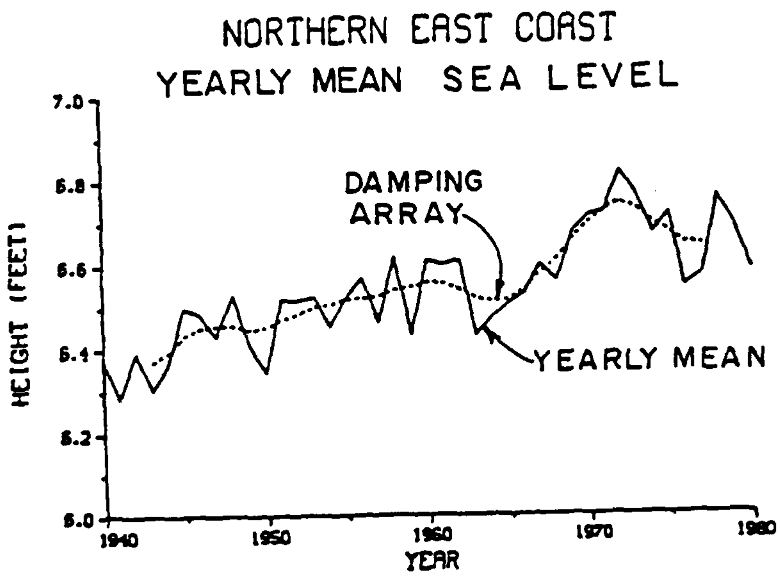
c. Frequency of Operation. An examination of tidal records at Boston for a 44-year period (NOAA Technical Memorandum NWS TDL 71, "A Tide Climatology For Boston, Massachusetts, November, 1982) indicates that the tide exceeded 7.5 on average about 5 to 6 times per year and wind and weather conditions indicate that, had the project been in place, it would likely have been operated on average, 2 to 3 times per year. Preparation for an operation may have been as much as 8 to 10 times per year. If tide levels continue to rise in the future this might necessitate more frequent operation or a change in target levels (refer to "Sea Level Rise" section).

14. RISING SEA LEVEL

a. Historic Rise. Sea level has been rising worldwide at varying rates for thousands of years. Since the maximum advance of the last glacier at about 13,000 B.C., sea level has risen between 330 to 490 feet (Shepard, 1963) or approximately 430 feet (Meade). With retreat of the glacial ice, the phenomenon of "rebound" of the landmass has accounted for more than 600 feet of increased elevation in northern areas of New England where the ice sheet was very thick. The mean height of the sea, with respect to the adjacent land, has been rising in the United States with the exception of Alaska and possibly northernmost New England where rebound may still be occurring. The overall long term historic rate of rise on the east coast has generally been 1 to 1-1/2 feet per century. This apparent change in sea level has been ascribed to a combination of increased water volume in the ocean from melting glaciers and subsidence of the land in some regions. Figure 20 depicts the historic relative sea level from 1940 to 1980 along the northern east coast (Hicks, 1983). At the Boston Harbor National Ocean Survey tide gage, the rise relative to the land has been estimated to be 0.008 ft/yr from 1922 through 1980. Sea level determination has generally been revised at intervals of about 25 years to account for the changing sea level phenomenon. The National Ocean Survey is presently finishing the process of reducing tide data from the 1960 to 1978 tidal datum epoch to make such a revision. Thus, the present local mean level of the sea at a given location along the coast can be expected to be several tenths of a foot higher than the National Geodetic Vertical Datum that was established as the mean sea level in 1929 and which remains basically fixed in time and space.

b. Future Sea Level Rise. In recent years there has been much discussion regarding a potential increased rate of future sea level rise. This phenomenon is related to a gradual warming of the earth's atmosphere associated with increased emissions of carbon dioxide and other gases on earth.

FIGURE 20



Northern east coast area mean with damping array.

Northern East Coast

Trend	2.6 mm/yr	.009 ft/yr
Standard Error of Trend	± .3 mm/yr	±.0011 ft/yr
Variability[c]	±24.5 mm	±.080 ft

Northern East Coast to Cape Hatteras

Portland, ME
Seavey Island, ME (Portsmouth, NH)
Boston, MA
Newport, RI
Willets Pt., NY
New York (The Battery), NY
Atlantic City, NJ
Baltimore, MD
Annapolis, MD
Solomons Island, MD
Washington, DC
Hampton Roads (Norfolk), VA
Portsmouth, VA

Source:
Sea Level Variations for the United States 1855-1980
National Ocean Service

The warmed atmosphere may promote expansion of near surface ocean water and increase the rate of melting of glaciers, thereby hastening the rate at which ocean levels appear to be rising. The scientific community is generally in agreement that the rate of global sea level rise will increase; however, there is lack of precision and agreement as to how much the increase will be. Several scientists have made projections employing mathematical models which simulate the processes involved. These global sea level rise forecasts by others are summarized in table 37. It can be seen that the increase in global sea level by 2075 could be about as little as 1 foot or as much as 7 feet. A middle estimate of 3 to 4 feet is accepted by many experts. This middle ground would yield an increase of nearly fourfold over historic rates in New England. The National Research Council (NRC, 1987) recently suggested that the sensitivity of design calculations and policy decisions be evaluated based on three plausible variations in sea level rise to the year 2100, all showing greater rate of rise in the distant future than in the next decade and all with an increased rate of rise relative to the present: 1.6, 3.3, and 4.9 feet. These estimates represent "Eustatic" or global changes. The local component which varies greatly from subsidence to uplift must also be included in estimating the total rise at a specific location. The NRC suggests the following equation for estimating total rise; $T(t) = ((0.0012+M/1,000)t+bt^2)/.3048$ in which $M = 1.0$ mm/yr at Boston, t = years from 1987, and $b = 0.000028$, 0.000066 or 0.000105 m/yr^2 for global rises of 1.6, 3.3, and 4.9 feet by the year 2100. The result here is converted to feet. Figure 21 presents NRC total plausible rise at Boston for the three cases.

c. General Corps Policy Regarding Sea Level Rise. The Corps policy regarding sea level rise is one of concern rather than alarm. The Corps is trying to stay aware of ongoing developments to further define the complex issue, keeping in mind the inherent uncertainty in any projections. A 21 March 1986 letter from the Office of the Chief of Engineers stated our policy as follows:

(1) Predicting future sea level rise is risky because there are so many variables and, as yet undefined interrelationships.

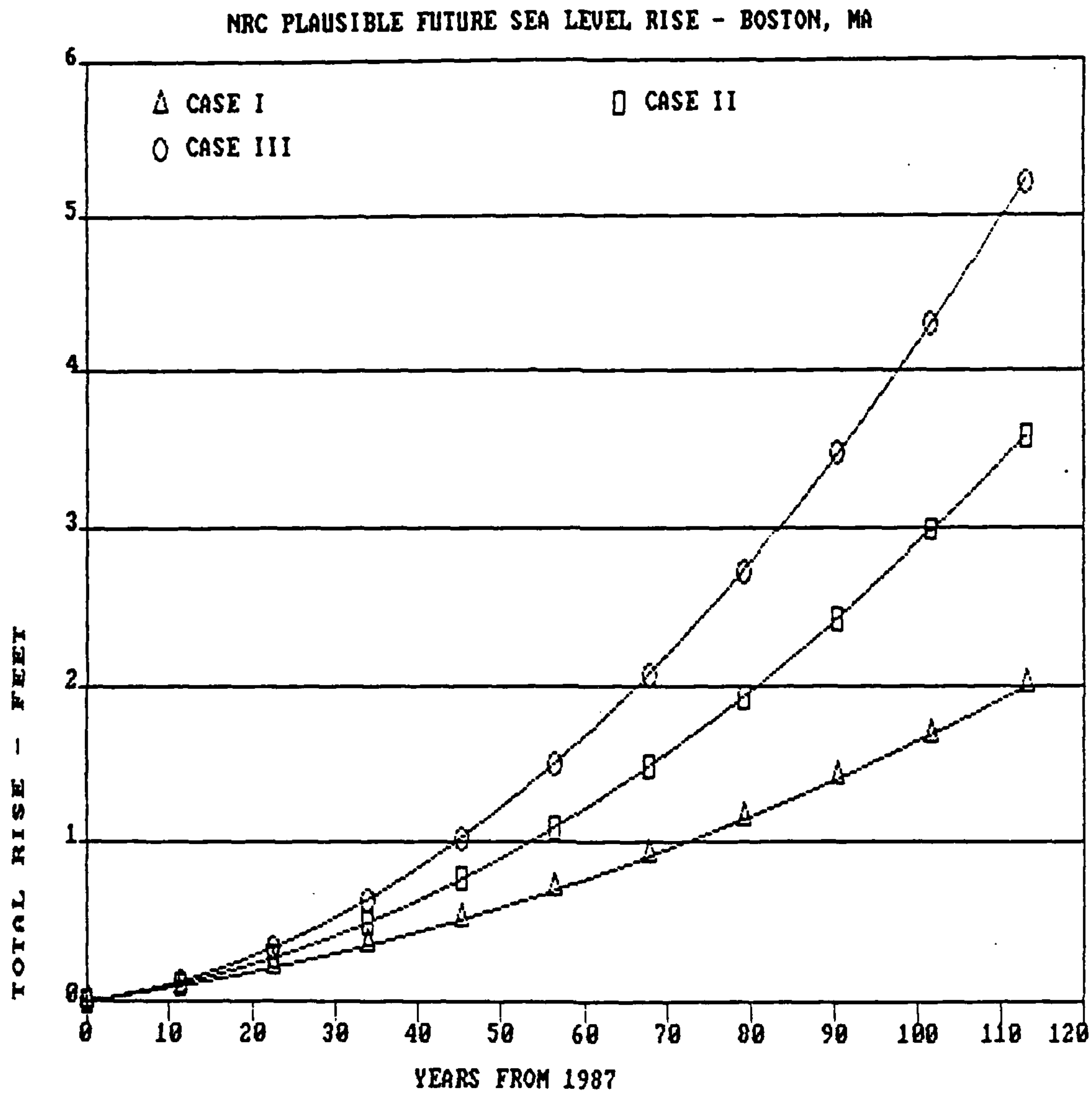
(2) Until substantial evidence indicates otherwise, we will maintain the procedure of considering only local regional history of sea level changes to project a rise or fall for a specific project.

TABLE 37

ESTIMATES OF FUTURE SEA LEVEL RISE (FEET)*

Investigator	Total Rise in Specific Years					
	2000	2025	2050	2075	2085	2100
Revelle (1983)*	-	-	-	-	2.3	-
Hoffman et al. (1983)						
Low	0.2	0.4	0.8	1.2	-	1.8
Mid-Range Low	0.3	0.9	1.7	3.0	-	4.7
Mid-Range High	0.4	1.3	2.6	4.5	-	7.1
High	0.6	1.8	5.8	7.0	-	11.3
Hoffman et al. (1986)						
Low	0.1	0.3	0.7	1.2	1.4	1.9
High	0.2	0.7	1.8	6.3	8.5	12.1

* NOTE: Other studies only provided an estimate for a specific year



PLAUSIBLE SEA LEVEL RISE FROM NATIONAL RESEARCH COUNCIL-1987

FIGURE 21

(3) Where long periods of tidal records exist and are used in determining the exceedance frequency relationship for coastal flood levels, it may be necessary to adjust the water level records for relative sea level changes when such changes are significant.

(4) Prudence may require an allowance in a project design for continuation over the project design life of an established significant long-term trend in relative sea level rise.

(5) Consideration must be given to the relative magnitude of the suggested allowance and the confidence band of the data the designer is using and the tolerance allowed in constructing the project.

(6) Consider whether it is more cost effective to include the allowance for significant sea level rise in the initial construction or to plan for modification later after the need for such is demonstrated.

As events continue to unfold and more precision is gained in estimating future sea level rise, additional Corps policy guidance is sure to follow. In an effort to make informed policy judgements the Corps Coastal Engineering Research Center is conducting an annotated bibliography on sea level rise. As well, plans have been made to embark on a study to determine the impacts of sea level rise on coastal engineering. On 20 June 1988, NED received draft Corps guidance relating to incorporation of sea level rise in feasibility studies. This guidance recommended a sensitivity analysis using historic and NRC case III projections.

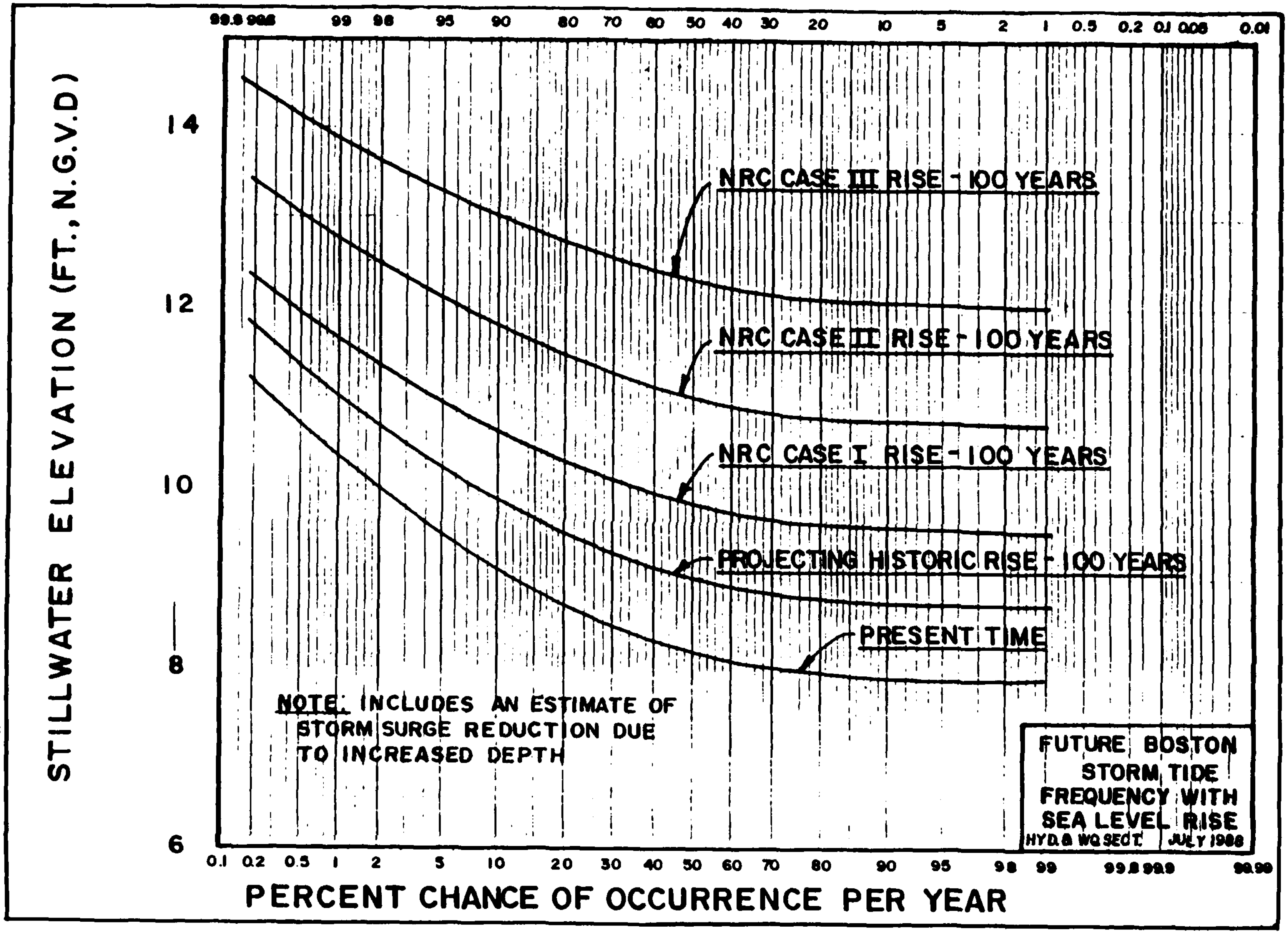
d. Effects of Rising Sea Level on Future Tidal Flood Frequency. Storm surges, the increased water levels induced by wind stresses and the barometric pressure reduction associated with hurricanes, tropical storms, and extratropical storms will be modified by sea level rise mostly in areas of very mild offshore slopes, as is typical of many southeastern states. The large expanse of shallow water resulting from higher sea levels will cause increased storm surge elevations, compared to areas of steep offshore slopes, because surge heights are proportional to both the length and inverse slope of the offshore bottom. However, if the shoreline is fixed and offshore water depths increase, as is typical of the study area, then the storm surges will be less, as the surge also varies inversely with absolute water depth (NRC, 1987). Reduction of the wind stress component of the storm surge can be estimated by the relationship of Dean and

Dalrymple (1984) where

$$\frac{\Delta n_{max}}{S} = \frac{(\Delta n_{max}/h_o)}{(1+n_{max}/h_o)}.$$

S = sea level increase, n_{max} = maximum wind stress storm surge, and h_o = a representative depth. By way of illustration, for a representative water depth of 30 feet, a 5-foot maximum storm surge (approximately the greatest observed at Boston) would be reduced by about 0.1, 0.2, 0.4, and 0.6 foot for respective sea level increases over the next 100 years of 0.8, 1.6, 2.9 and 4.2 feet, assuming the barometric component of storm surge is about 20 to 30 percent of the total. Lesser, more frequent, surges would be reduced a progressively smaller amount. Of course, relative to an absolute datum, the total flood elevation would increase by 0.7, 1.4, 2.5, and 3.6 feet, respectively. Figure 22 shows the natural Boston stillwater tide stage frequency curve as adjusted for continued historic rate of rise and for NRC cases I, II and III rates of rise over the next 100 years with the reduction in storm surge due to increased depth included.

Table 38 compares the frequency of tidal flooding in 2087 to that in 1987 assuming that the historic rate of rise of 0.08 foot per decade were to continue, accounting for the reduction of storm surge with depth as previously discussed. Even under these conditions today's 100-year flood could become about a 25-year event. Considerable flooding and resulting economic and environmental loss will surely be associated with increasing ocean levels.



STILLWATER ELEVATION (FT., N.G.V.D)
PERCENT CHANCE OF OCCURRENCE PER YEAR
NRC CASE III RISE - 100 YEARS
NRC CASE II RISE - 100 YEARS
NRC CASE I RISE - 100 YEARS
PROJECTING HISTORIC RISE - 100 YEARS
PRESENT TIME
NOTE: INCLUDES AN ESTIMATE OF STORM SURGE REDUCTION DUE TO INCREASED DEPTH
FUTURE BOSTON STORM TIDE FREQUENCY WITH SEA LEVEL RISE
HYD. & WQ SECT. JULY 1988
6
8
10
12
14
0.1 0.2 0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.8 99.9 99.99
99.9 99.8 99 98 95 90 80 70 60 50 40 30 20 10 5 2 1 0.5 0.2 0.1 0.05 0.01

FIGURE 22

TABLE 38

FUTURE FREQUENCY OF TIDAL FLOODING
BOSTON, MASSACHUSETTS

Average Return Period (years)	1987 Stillwater Elevation (ft, NGVD)	2087* Projected Stillwater Elevation (ft, NGVD)
10	9.1	9.8
50	10.0	10.7
100	10.3	11.0

* Based on projecting a historic rise in relative sea level of about 0.1 foot per decade, including estimated surge reduction due to increased depth.

It is recommended, in accordance with the previously referenced Corps policy, that the natural stillwater tide stage-frequency relationship for the Saugus River and Tributaries Study at least account for projection of the long term uniform historic rate of sea level rise (0.008 ft/yr) over the project life, approaching an increase in stage of 0.8 foot over a 100-year period. As well, qualitative consideration should be given to the possible future effects that accelerated rise (NRC cases I, II, and III) could have on the project and environs. We should not design for accelerated rise now, however, since any needed additional protection could be built over a relatively short period of time, once substantial sea level increase is confirmed (NRC, 1987). Rather, the sensitivity of the protective scheme should be qualitatively evaluated for several sea level rise scenarios, keeping in mind the probability of increasing sea level and keeping all response options open. Alternatives ill-suited for retrofitting should be avoided. The effectiveness of any flood protection scheme built today and subjected to significant sea level rise in the future would be a function of the project's durability and height. Obviously, significant sea level rise could cause greater wave or sea level overtopping as well as undermining of the structural integrity of any flood protection device. It is also important to assess the effects of sea level rise on the study area if no project is built.

e. Effects of Future Sea Level Rise on Tidal Flood Plain Zones

(1) General. It was requested that the effects of future sea level rise on existing natural and modified stage frequencies for the various flood plain zones be examined. This was to be used by others to determine approximate economic benefits, attributable to the project, due to future rising sea level. As previously discussed, estimates of future rate of rise vary considerably. However, Corps of Engineers policy is to consider only local regional history of sea level changes in determining economic benefits. Therefore, the historic rate of rise of about 0.1 foot per decade was used for this cursory economic assessment.

(2) Effects on Existing Boston Stillwater Curve. Recognizing the difficulties in trying to precisely determine the impacts of a future gradual rise in sea level on existing stage-frequency relationships it was decided, for simplicity sake, to estimate stage-frequency relationships at a distant future time (say 100 years hence). Since the historic rate of rise is about 0.1 foot per decade, the future sea level condition elevation frequencies would be the existing condition elevations plus approximately 1 foot. It is noted that this assumption is not precisely correct as discussed in section 8d; however, it is felt that it is adequate for the current investigation. Thus, based on the above assumptions, the adopted future sea level condition Boston stillwater elevation frequencies would be as follows in 100 years.

Percent Chance Occurrence	Existing Condition Elevation (ft, NGVD)	Future Condition (100 Years) Elevation (ft, NGVD)
SPN	12.0	13.0
0.2 (500-yr)	11.2	12.2
1.0 (100-yr)	10.3	11.3
10 (10-yr)	9.1	10.1
50 (2-yr)	8.3	9.3

(3) Effects on Natural Stage Frequencies for Interior Zones. For interior zones (i.e., those not directly bordering the estuary) where the Boston stillwater curve is not directly applicable, several techniques were explored in attempting to determine future sea level condition natural curves. First, as with the Boston stillwater curve, it was assumed to simply add 1 foot to all previously developed

natural stage frequency elevations. Second, noting that most zones have experienced 1978 flood elevations available, it was decided to assign the respective experienced 1978 interior flood elevations the frequency of occurrence that the 1978 flood ocean stillwater level would have on the future sea level condition stillwater curve (about a 7 percent chance). This single point in general was slightly higher than; however, compared well with the curves developed in step one. Also, it is felt that for the extremely rare frequencies interior flood levels (which are related to wave overtopping and local drainage) would tend to reach some maximum elevation and most likely would not exceed elevation 14 to 15 feet NGVD for a 1 foot rise in sea level over the next 100 years. For simplicity sake and because of the numerous uncertainties (i.e., the magnitude of sea level rise) it was decided to use existing interior curves plus 1 foot. This approach will allow reasonable estimates of the sensitivity of the area to continued historic sea level rise.

(4) Effects on Project Modified Stage-Frequencies. Trying to quantify the effects of future sea level rise with a tidal floodgate project is more difficult. A 1-foot rise in sea level would have effects on resulting waves and wave overtopping. Attempting to quantify such effects, is considered beyond the scope of current studies. A simplified means of determining future sea level condition project modified curves was developed as follows:

(a) The existing sea level condition and future sea level condition (existing plus 1 foot) natural relationships were analyzed.

(b) For a given frequency of occurrence, the existing sea level condition project modified elevation was determined (i.e., 1 percent chance, project modified elevation of 7.4 feet NGVD) reference table 17.

(c) This project modified elevation was then assigned a future sea level condition frequency based on the frequency shift of the natural curve from existing to future sea level conditions. In the case of the existing natural 1 percent chance flood its future frequency of occurrence would be about 7 percent. This was repeated for a range of frequencies to produce the future sea level condition project modified curves. Typical estimated future sea level condition natural and project modified curves are shown on plate 8. Estimated future condition elevation frequencies with and without project (SPN Design Level) are shown in table 39. Estimated future condition elevation frequencies with and without protection at Point of Pines are shown in table 39A.

TABLE 39

REVERE, MASSACHUSETTS
ESTIMATED FUTURE FLOOD STAGE FREQUENCIES
(Feet NGVD)

Location - Condition		Annual Frequencies (%)			
	0.2	1.0	10	50	90
Future Sea Level Condition, Boston Stillwater (Existing + 1 Foot)	12.2	11.3	10.1	9.2	8.9
Zone 1 - Estimated Future Sea Level Natural	9.6	8.1	6.0	4.8	4.5
Modified By Tidal Protection	9.6	8.7	4.4	3.8	3.5
Zone 2A - Estimated Future Sea Level Natural	14.0	12.3	8.6	7.5	7.4
Modified By Tidal Protection	6.8	6.7	6.6	6.5	6.4
Zone 2B - Estimated Future Sea Level Natural	11.9	10.3	6.8	4.8	4.4
Modified By Tidal Protection	4.8	4.6	4.4	3.8	3.4
Zone 3A - Estimated Future Sea Level Natural	10.0	7.1	5.0	4.4	4.1
Modified By Tidal Protection	9.4	6.5	5.0	4.4	4.1
Zone 4A - Estimated Future Sea Level Natural	10.8	9.3	6.5	4.8	4.4
Modified By Tidal Protection	4.8	4.6	4.2	3.8	3.4
Zone 4B - Estimated Future Sea Level Natural	9.4	7.5	4.8	4.2	4.0
Modified By Tidal Protection	3.7	3.5	3.3	3.2	3.0
Zone 4C - Estimated Future Sea Level Natural	10.5	8.7	6.8	6.2	6.0
Modified By Tidal Protection	9.6	5.4	5.3	5.2	5.0
Zone 5A - Estimated Future Sea Level Natural	12.2	11.3	10.1	9.2	8.9
Modified By Tidal Protection	9.3	7.7	7.3	7.2	7.1
Zone 5B - Estimated Future Sea Level Natural	12.2	11.3	10.1	9.2	8.9
Modified By Tidal Protection	12.0	10.0	9.2	7.3	7.0
Zone 5C - Estimated Future Sea Level Natural	12.2	11.3	10.1	9.2	8.9
and 5D Modified By Tidal Protection	9.3	7.7	7.3	7.2	7.1
Zone 6 - Estimated Future Sea Level Natural	12.8	11.7	10.0	8.9.	8.4
Modified By Tidal Protection	9.3	7.7	7.3	7.2	7.1

TABLE 39 (cont)
LYNN, MASSACHUSETTS

Location - Condition	0.2	1.0	10	50	90
Future Sea Level Condition, Boston Stillwater (Existing + 1 Foot)	12.2	11.3	10.1	9.2	8.9
Zone 1 - Estimated Future Sea Level Natural	14.6	13.4	11.1	10.0	9.5
Modified By Tidal Protection	10.5	8.6	8.5	8.5	8.4
Zone 2 - Estimated Future Sea Level Natural	13.0	12.2	10.9	10.1	9.8
By Tidal Protection	10.4	8.7	8.5	8.5	8.4
Zone 3 - Estimated Future Sea Level Natural	12.2	11.3	10.1	9.2	8.9
Modified By Tidal Protection	9.3	7.7	7.3	7.2	7.1

SAUGUS, MASSACHUSETTS

Location - Condition	0.2	1.0	10	50	90
Future Sea Level Condition, Boston Stillwater (Existing + 1 Foot)	12.2	11.3	10.1	9.2	8.9
EAST SAUGUS					
Zone 1 - Estimated Future Sea Level Natural	13.1	12.0	10.2	9.2	8.9
Modified By Tidal Protection	8.5	7.5	7.2	7.1	7.1
Zone 2 - Estimated Future Sea Level Natural	12.9	11.7	9.3	7.0 (21%)	6.0 (30%)
Modified By Tidal Protection	9.0	6-	-	-	-
Zone 3 - Estimated Future Sea Level Natural (Upper Limit)	12.7	11.5	9.6	8.0	7.8
Modified By Tidal Protection	8.0	7.6	7.2	7.1	7.0

TABLE 39A

REVERE, MASSACHUSETTS
ESTIMATED FUTURE FLOOD STAGE FREQUENCIES
POINT OF PINES
(Feet NGVD)

Location - Condition	Annual Frequencies (%) 0.2	1.0	10	50	90
Future Sea Level Condition, Boston Stillwater (Existing + 1 Foot)	12.2	11.3	10.1	9.2	8.9
Zone 1 - Estimated Future Sea Level Natural	14.8	14.0	12.0	11.0	10.8
(7A) Modified by Tidal Protection	14.8	13.0	9.8	9.7	9.6
Zone 2 - Estimated Future Sea Level Natural	13.6	13.0	11.2	9.6	9.2
(7B) Modified by Tidal Protection	13.6	12.0	7.8	7.8	7.7
Zone 3 - Estimated Future Sea Level Natural	12.0	11.0	9.6	8.8	8.5
(7C) Modified by Tidal Protection	12.0	10.5	7.2	7.2	7.2
Zone 4 - Estimated Future Sea Level Natural	12.0	10.8	9.0	8.0	7.9
(7D) Modified by Tidal Protection	12.0	10.0	6.3	6.2	6.2

f. <u>Effect of Rising Sea Level on Tide Ranges and Currents</u>. The increase in sea level in many sheltered embayments will be felt predominantly through an increased water level. The depth increase will facilitate the propagation of tidal waves due to depth dependence. Many areas which have sedimentation rates of the same order as the relative sea level rise will notice minimal change in tidal characteristics. Since sediment input to the Saugus and Pines Rivers Estuary is limited, it is felt that increased sea level will bring the estuarine tidal regime into closer phase with that in the open coast. Since most of the area, except the Sea Plane Basin (plate 9), is now nearly in phase these changes will generally be minimal.

The tidal prism may be substantially increased if significant sea level rise occurs since the Saugus Marsh area which is now at about mean high water will routinely be flooded to much greater depth than the present. Also shoreline retreat could further increase the prism volume. The deeper water will also increase prism exchange by reduction of friction at the tidal entrance due to deeper water. O'Brien (1969) has shown a relationship between increase in tidal prism and increase in cross sectional area of a sandy tidal inlet, indicating they are directly proportional. Table 40 developed from the existing capacity curve for the Saugus and Pines estuary shows potential changes in mean and maximum tidal prism or flushing volume for various cases of sea level rise. In actuality, erosion may significantly alter future tidal volumes. Tidal currents would be expected to increase similarly to the tidal prism. Additionally, the change in tidal prism, assuming the proposed floodgate would be closed for astronomic tides exceeding 7.5 feet, NGVD, has been included.

This would indicate that under natural conditions the entrance to the Saugus River could begin a somewhat expansionist tendency which may at some point require evaluation of some measures to control erosion on the banks and channel scouring which may cause problems at the existing bridge overpass. If the proposed tidal floodgate is constructed across the Saugus River mouth, this problem may be somewhat reduced since the river will be constrained to flow through the concrete structure. However, with very large sea level rise, currents during mean tides through the gated openings could increase dramatically, possibly necessitating evaluation of additional measures, such as locks or more gates. This should not be as much of a problem if the N4 or EN schemes are adopted now. Floodgate frequency of operation would probably be increased since many inshore areas would be flooded by normal high tides. Table 41 shows the sensitivity

TABLE 40

CHANGE IN TIDAL PRISM

Scenario	2087 Sea Level Rise (feet)	Natural Percent Increase in Tidal Prism Using Existing Estuary Capacity (mean/maximum)	Percent Change in Tidal Prism with Floodgate Closure at 7.5 Feet NGVD (mean/maximum)
Projected			
Historic Rise	0.8	9/15	+ 9/-2
NRC Case I	1.6	22/34	+22/-4
NRC Case II	2.9	56/60	+42/-8
NRC Case III	4.2	100/80	+29/-14

of floodgate closure to sea level rise assuming that no localized protective works or relocations are undertaken. This table is based on data from the NWS "Tide Climatology for Boston, Massachusetts" which includes an adjustment for historic rising sea level. A rise of 2 feet or more could necessitate consideration of locks and pumps due to the long period each year when the gates would be closed. Water quality during closure could also be adversely affected. As an alternative, the start of damage could be elevated by constructing/raising walls and/or dikes around the estuary.

Additionally, tides of the greater Boston area are part of the Gulf of Maine system whose tidal dynamics are near resonance. Small changes in sea level could have significant effects on the ocean tidal heights and currents by altering this resonant condition. The threat of large sea level rise could make further research on this issue a reality.

TABLE 41

FLOODGATE CLOSURES WITH SEA LEVEL RISE
(No Change in Start of Damage)

Sea Level Conditions	Average Annual Number Of High Tides > 7.5' (ft, NGVD)	Average Annual Number Of Floodgate Closures	Typical Duration of Gate Closure (hours)
Today	6	2-3	1-2
1 foot rise	80	35-45	2-3
2 foot rise	400	175-225	3-4
3 foot rise	625	400-450	4-5
4 foot rise	675	575-600	5-6

g. Effects of Rising Sea Level on Waves . Two differing phenomena need to be considered when examining the effect of increased sea level on wave propagation. First, when waves are generated in deep ocean waters and progress shoreward over the Continental Shelf, the waves are dampened. This dampening is related to the width of the shelf and depth of water. The physics involved are quite complex. The NRC (1987) has shown as an example (Dean and Dalrymple, 1984) that for a depth of 33 feet, shelf width of 6.2 miles, wave period of 8 seconds, initial wave height of 6.6 feet, friction coefficient of 0.01 and sea level rise of 3.3 feet, a

0.2 foot or about 3 percent increase in wave height would be expected. The NRC indicates that this small increase is not likely to cause changes of substantial engineering significance.

The second case involves waves which are generated by the wind as it passes across the Continental Shelf waters. Here, wave growth will be enhanced by the deeper water because of the diminished effect of bottom friction. This effect can be estimated for the case of very long fetch using the Shore Protection Manual (Corps of Engineers, 1984), shallow water wave forecasting relationship. Simplifying,

$$\Delta H = (0.75 \frac{H}{h})S$$

where ΔH = change in wave height, S = increase in water depth, H = original wave height, and h = representative water depth. For the same values of the previous example, wave height would increase about 0.5 foot or 7.5 percent.

The combined effect of reduced wave dampening and augmented wave generation would result in larger wave heights in the surf zone. Larger amounts of sediment would be moved and greater wave forces and potential for overtopping would exist. In the coastal area of the project this could mean accelerated beach erosion along Revere Beach, Point of Pines, and Nahant Beach. With excessive sea level rise and no continuing maintenance, breaching of barrier or causeway beaches cannot be ruled out as well as undermining and failure of existing seawalls and revetments. Minor sea level rise, in the order of historic rates, will at least create greater wave overtopping and flooding behind coastal structures. The amount of impact will increase as sea level rise increases. Major structural actions or abandonment may be necessary in the future in the study area and all along the United States Coast, if large accelerated sea level increase occurs. Local assurances for the project should require that local interests maintain the present configuration of protection along Revere Beach, Point of Pines and the Nahant Causeway.

h. Effects of Rising Sea Level on the Coastal Zone. Mean sea level is one of the primary factors determining shoreline position along sandy coastlines such as exist in the study area. It has been suggested by Swift et al. (1972) that a relationship exists between sediment supply, wave energy, sea level and shoreline position. Rising relative sea level will tend to promote shoreline recession, except when the influx of sediment can offset this recession. The present eroded state of Revere Beach is testimony to the argument that sediment influx is insufficient under present conditions to offset shoreline recession and that under future accelerated sea level rise things will just get worse. The link between shore retreat and sea level rise based on examination of beaches both developed and undeveloped, worldwide, indicates that the relationship is causal in nature (NRC, 1987). However, in some locations, man's activities relating to construction of seawalls, jetties, etc. have probably increased shoreline recession.

Bruun (1962) was the first individual to formulate a relationship between rising sea level and the rate of natural shoreline erosion. His basic premise is that each beach will try to maintain an "equilibrium profile," where material removed during shoreline retreat is transferred onto the adjacent inner shelf, thus maintaining the original beach profile and near shore shallow water conditions. Of course this concept is mainly applicable to natural beaches rather than those protected by seawalls as exist in the study area. However, for comparative purposes examination of Bruun's concept is warranted. In general, Bruun's relationship shows that for a sea level rise S, the shoreline retreat will be about 100S. Table 42 presents possible shoreline retreat for varying increases in sea level.

TABLE 42

NATURAL BEACH RESPONSE TO RISING SEA LEVEL

Scenario	2087 Sea Level Increase (feet)	Shoreline Retreat by Bruun Relationship (feet)
Projected Historic Rise	0.8	80
NRC Case I	1.6	160
NRC Case II	2.9	290
NRC Case III	4.2	420

It should be noted that areas like the Nahant Causeway, Revere Beach, Point of Pines will likely be subjected to greater erosional forces as indicated by Bruun's relationship. Erosion will occur on beaches and at the base of seawalls and revetments, possibly causing undermining leading to failure, unless stringent maintenance practices are followed. Breaching of beach barriers cannot be ruled out. Major structural actions or abandonment may be required to hold back the sea with major sea level rise. Breaching of the Nahant Causeway could seriously threaten Lynn Harbor which is now protected from deep ocean waves. The degree of shoreline erosion is closely related to the amount of sea level rise. Continued historical rate of rise will see a minor increase in erosion. Whereas, accelerated rise (NRC cases I, II, III) could see progressively accelerating erosion. Significant expenditures will likely be necessary if the effects of a large increase in sea level are to be curbed. The project local assurances should require that local interests maintain existing protection along Revere Beach, Point of Pines and the Nahant Causeway.

Holding back the sea as water levels rise will almost always be technically feasible; however, in some cases it may not be economically or environmentally sound. Without political or emotional considerations, economics will be the final arbiter in deciding whether or not to retreat (NRC, 1987).

i. <u>Hydraulic Effects of Rising Sea Level on Tidal Estuary</u>. The intent of this section is not to address the biological implications of sea level rise (see Environment Impact Report), but rather to speak of hydraulically induced physical changes which will alter the present estuary regime. Previously the potential for significant increase in tidal prism was discussed. As the prism increases, shallow bays may expand rapidly in response to a rise both because of gentle slope and erosion of adjacent land areas in response to water level increases (NRC, 1987). As well, if the prism increases, there is likely to be a corresponding increase in the volume of shoals, the increase in shoal volume being related to erosion of the surrounding land areas. In the case of the Saugus Marsh, its perimeter is already bordered by intense urban development which itself would be subject to possible routine inundation with significant sea level rise. In the absence of increased flood protection many low lying urban areas may be abandoned allowing gradual expansion of the tidal flood plain area. Walls/dikes could also be built to prevent flooding and thus prevent flood plain expansion. Geise (1987) has recently quantified potential passive loss of these upland areas in Massachusetts for various scenarios of sea level rise. His results for Lynn, Saugus and Revere are shown in table 43. With improved flood protection, flood

TABLE 43

CALCULATED UPLAND RETREAT
(Areas are in acres, % represents percent of upland submerged)

Town Name	Upland Area Acres	Historical Annual Retreat 0.01 Ft/Yr Rise		Total Retreat: 1980 - 2025 Case 1 0.45 Foot Rise		Case 2 1.14 Foot Rise		Case 3 1.57 Foot Rise	
		Percent	Area	Percent	Area	Percent	Area	Percent	Area
Lynn	6,336	0.004	0.26	0.18	11.7	0.47	29.6	0.64	40.8
Revere	2,595	0.009	0.24	0.41	10.7	1.05	27.2	1.44	37.5
Saugus	5,859	0.002	0.13	0.10	6.1	0.26	15.4	0.36	21.2

plain expansion would be curbed. Although generally, most estuaries do receive adequate sediment supply to compensate for current sea level rise, it is felt that upstream development already significantly limits freshwater sediment supply to the area. With significant increase in sea level rise, sediment replenishment will likely be even less significant.

Near the present upstream limit of tidal effects on the Saugus River, rising sea levels will promote saltwater intrusion. The tide gate of the Pines River will limit upstream progression of tidal influence there.

The mechanism of loss of flood plain lands with a large increase in sea levels will be the formation of extensive interior ponds allied with general tidal creek bank erosion and headward growth as tidal prisms increase (NRC, 1987). Further discussion of related biological factors is presented in the Environment Impact Report.

j. <u>Effect of Rising Sea Level with Flood Protection Measures</u>. Flood protection measures are generally built to provide protection up to some design flood event. Frequently this is the 100-year flood. The height of the flood control structure is usually set several feet above this design flood stillwater level to allow a factor of safety and to substantially minimize any wave runup and overtopping. Increased sea level will tend to reduce the factor of safety over the project life and allow for gradually increasing wave runup and overtopping. Past New England Division policy has been to allow 2 or 3 feet above design stillwater level for a factor of safety, on concrete walls and earthen dikes, respectively, where minimal wave action is expected. Where significant wave action is likely, the height is increased to reduce wave overtopping to manageable amounts (see paragraph 11b). It has been recommended that a height allowance of no less than 2 feet be added to design stillwater level for all Saugus and Pines Rivers area tidal protection which is designed to withstand some degree of wave overtopping. The height allowance should be increased to a minimum of 3 feet if the protection is not designed for overtopping. At some future date, additional studies may need to be conducted to determine if structure height needs to be increased due to accelerated sea level rise and increasing wave overtopping. In order to maintain the design level of protection, height increases would be at least equal to the sea level rise.

The operation of any flood protection scheme will also be impacted by sea level rise. In the case of "separate local protection projects" the closure of flap gates and ponding of interior runoff will become more frequent. Large increase in sea level might necessitate the addition of pumping stations to pump interior runoff through the line of protection, increased sea level having minimized the effectiveness of gravitational discharges.

The Regional Saugus River Floodgate Plan will also experience changed operation due to large increase in sea level. As presently formulated the floodgates would be closed only a few times per year when storm tides are expected above +8.0 feet NGVD, this being the approximate start of flood damage. Normal tides, nearly all the time, will be allowed to pass virtually unobstructed through the open gates. However, with large sea level rise, normal nonstorm high tides could frequently cause flood damage throughout the area. This could necessitate very frequent closure of tidal floodgates during nonstorm periods to protect low lying residential areas in the future if localized protection or relocation is not implemented (see table 41). Frequent floodgate closure in this instance would minimize water depths during frequent periods of prolonged estuary inundation. Additionally, increased wave overtopping entering the backshore ponding area may necessitate earlier closure of tidal floodgates during storms to assure that ample ponding area is available. The large amount of storage available makes it unlikely that large sea level rise would mandate the addition of pumps at the floodgate structure to carry interior runoff although this option cannot be ruled out.

As previously stated currents through the gated openings would increase with a large increase in sea level (table 40). Studies may be necessary in the future to address this potential problem possibly through the use of locks or additional gated openings. As well, sedimentation in the river channels would be expected to increase with increased erosion in the tidal marsh area. Erosion prompted by large sea level rise may necessitate significant maintenance activities throughout the protected area to assure continued flood protection, especially along Revere Beach itself.

k. Perspective. In the preceding sections, historical sea level rise and National Research Council scenarios of potential increased future sea level rise have been discussed. Possible increases in flood frequency, tidal ranges and currents, waves and coastal erosion have been explained. The significant hydraulic erosional threat to tidal flood plains has been identified. Flood protection measures constructed today may be affected by large sea level rise. This may increase operation and maintenance costs in years to come and may necessitate additional future studies to increase project effectiveness.

It is important to realize that large sea level rise would be a worldwide problem with many major urban centers being faced with potential significant flooding and other problems. Although discussion has focused on the project area, the effect on a national level would be huge. Major economic resources would be required throughout the country to deal with this problem if it materializes.

The best course of action at present is not to panic but stay aware and closely monitor the situation and be prepared to act when increased rise can be predicted with high confidence.

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* Includes references not specifically spelled out in report

ADDENDUM I

CONSTRUCTION SEQUENCE
HYDRAULIC CONSIDERATIONS

The proposed construction sequence for the project is fully described in the Design and Cost Appendix. In summary it involves: (1) dredging a temporary bypass navigation channel and dredging to finish grade at flushing gate locations; (2) installing ring cofferdam and building navigation gate; (3) concurrently, installing braced cofferdam and building first four tainter gates and gravity wall on north (Lynn) side (mid-tide flow area of about 5,500 square feet maintained until braced cofferdams removed); (4) with four completed tainter gates and navigation gate open, construct braced cofferdam and build south (Revere) tainter gate; (5) concurrently, install braced cofferdam for remaining five tainter gates (5,700-square foot flow area maintained below mid-tide); (6) completed south (Revere) tainter gate is opened and Revere gravity wall and dike are constructed (5,300 square feet of flow area maintained below mid-tide level); (7) all cofferdams removed and all gates open for remainder of construction (8,800-square foot flow area below mid-tide level).

Since at least 5,200 square feet of flow area will be maintained below mid-tide at all times, it is expected that hydraulic effects will be similar to that for the N3 scheme described in table 25. Average mid-tide currents will be about 3 feet per second, reaching up to just over 5 feet per second during extreme spring tides. Basin tide levels will not appreciably change although a very small (0.5 to 3 percent) reduction in tidal flushing may temporarily occur.

The preliminary CEWES proposed mathematical hydrodynamic modelling (attachment 2) includes a future evaluation of local current patterns and flushing to be expected during the construction sequence. Model results will allow refined evaluation of riprap requirements for protection of cofferdams and shoreline property from any possible scour and erosion during construction.

ADDENDUM II

PROPOSED HYDRAULIC AND NAVIGATION MODEL STUDIES

In response to comments raised at the Issue Resolution Conference held 18-19 October 1988, CENED formally requested a proposal, cost estimate and schedule from CEWES relative to future model studies. Pending a future meeting between CENED, CEWES, and HQUSACE to refine the modelling proposal, CENED has provided CEWES some comments on the proposal. After the draft feasibility report has been sent to HQUSACE for review in the spring of 1989 and funds become available, refinement of the modelling scope will commence. This addendum contains correspondence in reference to the above.

DISPOSITION FORM

For use of this form, see AR 340-15; the proponent agency is TAGO.

REFERENCE OR OFFICE SYMBOL	SUBJECT
CENED-ED-WQ	Model Study Meeting - Saugus River Floodgate

TO	FROM	DATE	CMT 1
Memo for the Record	Chief, Hydraulics & Water Quality Section	10 Mar 89 Mr. Wener/mbc/7686	

1. On 1 March 1989, the following individuals met at the Waterways Experiment Station (WES) to discuss modelling requirements for the Saugus River Floodgate Project, specifically the WES proposal of 27 January 1989 and NED's letter expressing concerns, dated 7 February 1989 (copy attached). Mr. Wener was at WES on other business at the time and it seemed useful to meet to further discuss NED's concerns relative to the Saugus River Project.

<u>WES</u>
Bill Martin - Estuaries Div, Estuarine Eng Br
Chris Hewlet - Waterways Div, Navigation Br
Carl Huval - Waterways Div, Navigation Br
Glen Pickering - Hydraulic Structures Division
<u>NED</u>
Chuck Wener - Hydraulics & Water Quality Sec

2. The meeting began with a general discussion of the project, then focused on concerns raised in the 7 February 1989 NED letter. These discussions are summarized as follows in order of appearance in the NED letter.

a. Concern discussed, no comments.

b. Construction phases were discussed and WES will revise appropriately.

c. WES indicated that the construction sequence will likely be the critical period for flushing and navigation. It will be important to address this period in the model. The recent monetary settlement between the Massachusetts Bay Transportation Authority (MBTA) and the lobstermen and General Electric regarding the inability to open the Saugus River commuter rail bridge was discussed. We may be subject to claims if we change navigation conditions during construction.

d. It appears reasonable to use the 13-foot tide range for design related to navigation currents. However, the maximum 14.7-foot range may be more appropriate for riprap/structural design. The normal spring tide range (11 feet) may need to be used in initial mathematical model setup and calibration for reasons of computational stability.

e. It was generally agreed that mathematical sediment modelling could be eliminated. General sediment patterns could be identified using "glass beads" in the physical model at less cost and accuracy.

f. NED can use WES design criteria for sizing riprap. However, if riprap is tested in the physical model, less and/or smaller riprap may be justified since design criteria is generally conservative.

g. The physical model can be used to develop discharge coefficients for the gates including the effect of flared entrances and rounded corners. Otherwise, conservatively low coefficients must be assumed. The difference in required gate size could be significant. As well, the physical model can determine near field currents 3-dimensionally through the structure itself. These are the currents the vessels will feel. The mathematical model cannot really determine these currents at the structure.

h. WES feels that "tow tank tests" may be rightfully chargeable to the project. "Upgraded computer technology" may not be needed if the design vessel is the General Electric fuel barge. WES did not feel that a "desk top" study could be done. The possibility of using a small scale electric boat in the physical model to do navigation tests was discussed. This approach may cost much less and take less time. <u>Further discussion of navigation requirements is needed to resolve this issue.</u>

i. Tide and current data must be synoptic to be used for model calibration; therefore, past data likely can't be used. However. NED could provide boats and labor to assist in future data collection.

j. It was noted that preliminary navigation model results would be available in May 1991 and final report completed in November 1991. This means that preliminary results would be available in the 20th month of study. WES has revised scheduling charts to be consistent.

k. Concern discussed, no comment.

l. The size of the gate openings may be refined during the model studies. Structural people should look at protection of gates in saline environment. Also, WES felt that there may be problems closing miter gates during maximum currents. This should be closely examined in design. Also, miter gates can easily be damaged by vessel impact. Miter gates can't stand much reverse head.

m. Mathematical sediment modelling will likely be eliminated.

n. Concern discussed, no comment.

o. Concern discussed. This item will not likely be part of WES' scope of work.

p. Progress reports and meetings will be included.

3. It should be noted that further discussions are still needed to finalize the WES scope of work subsequent to submittal of the draft planning report to OCE. Any additional OCE comments can also be discussed at that time.

Charles J. Wener

Atch

CHARLES J. WENER
Chief, Hydraulics &
Water Quality Section

CF:
Attendees at meeting

CENED-ED-WQ (CENED-ED-WQ/15 Dec 88) 2d End Mr. Wener/ dmp/ 617-647-8686
SUBJECT: Model Study - Saugus River Flood Control Project

Commander, New England Division, Corps of Engineers, 424 Trapelo Road, Waltham, MA 02254-9149 7 February 1989

FOR Commander and Director, Waterways Experiment Station, ATTN: CEWES-HE-E/Mr. William D. Martin, P.O. Box 631, Vicksburg, MS 39180-0631

1. New England Division has reviewed your proposal for comprehensive model studies for the Saugus River Flood Control Project. Generally we find the proposal to be well thought out and prepared in a very professional manner, especially in light of the short timeframe allocated for your efforts.

2. We agree that future meetings must be held between CEWES, CENED, and HQUSACE personnel to further refine the proposal to our mutual satisfaction. The estimated cost is especially sensitive due to the local sponsor cost sharing requirement. A meeting might be appropriate after the draft feasibility report is submitted to HQUSACE in the spring of 1989. At that time and as funds become available, we will initiate the refinement process. In the interim, several thoughts on the proposal are presented for future consideration.

a. Page 1, paragraph 3. The floodgate proposal as presently formulated is believed to produce negligible changes in the tidal regime and flushing/circulation within the estuary and adjacent ecologically significant wetlands. Any future optimized proposals must meet the same objectives to have a high probability of being implemented.

b. Page 6, Construction Phase Tests. This section should indicate that the construction under Phases 1A and 1B will be simultaneous, with the cofferdam being removed for each gate structure (e.g. - miter gate and Lynn tainter gates) when construction of that respective structure is finished.

c. Page 6, paragraph 1. Since it is "doubtful that any" of the construction phases "will provide problems and require further testing", can this aspect be deleted from the model study?

d. Page 6, paragraph 2. It is questioned if the "normal spring tide range" (mean spring tide range equals 11 feet) is appropriate as the design tide range since it is exceeded by 20 percent of all tide ranges. The tide range which is exceeded by 5 percent of all ranges (about 13 feet) may be more appropriate as the design range. This may be significant since the tide range has a relatively large realm of variability.

e. Page 6, paragraph 3. Since "the area has not experienced shoaling problems and it is not likely that the project will cause new problems", can sediment transport modeling be eliminated from the study?

f. Page 6, paragraph 4. Can riprap design be accomplished by CENED using hydrodynamic model results and current CEWES criteria?

g. Page 7, paragraphs 1 and 2. Previous discussions regarding modeling requirements for the project indicated that mathematical hydrodynamic analysis should be sufficient. Is the additional accuracy gained in the physical model now being proposed worth the added expense?

h. Page 7, paragraph 4. The navigation model would require "additional development activities" to include "tow tank tests" and "upgraded computer technology". It does not seem appropriate to ask the local sponsor to cost share in research and development activities. In lieu, could a scaled down "desk top" navigation study be conducted to establish a reasonable design width, tidal range, and maximum local velocity criteria for the navigation opening?

i. Page 9, paragraphs 1 and 2. Could the data collection program be scaled down to make use of tide and current data gathered previously by NED? This tide data was used in storm surge modeling by CERC in technical report, CERC-86-8. As well, NED could assist in future data collection, possibly reducing costs.

j. Page 12, Schedule. Enclosure 2 indicates that both mathematical hydrodynamic and physical modeling will be completed within the first year of model studies. The navigation model consumes principally the second year of study.

If a scaled down "desk top" navigation study were to be performed up front, the total CEWES effort could be curtailed to about one year. Additionally, enclosures 1 and 2 show riprap design being completed prior to erosion evaluation. Enclosure 3 does not agree with enclosures 1 and 2.

k. Page 12, paragraph 3. It is understood that contingencies presented include only hydraulic considerations related to the proposed modeling and do not address other hydraulic, mechanical, structural, etc., factors.

l. Page 13, paragraphs 1 and 2. It is understood that the navigation opening may need to be increased while the flushing opening might be decreased.

m. Page 13, paragraph 3. Since maintenance dredging is not expected to increase, can sediment modeling be eliminated?

n. Page 13, Results, d. The project must be formulated to have negligible effect on normal water circulation and elevations within the estuary to have a high likelihood of being built.

o. Page 13, Results, j. To have a high probability of being environmentally acceptable and being constructed, there must be negligible change in areas normally inundated landward of the structure.

p. Page 14. Since the comprehensive study would span 25 months, should monthly progress reports and periodic progress review meetings (either at WES or NED) be included?

3. In addition, although the model studies are not being conducted for environmental purposes, in order to accomplish good hydraulic design, the designer/modeler should be conscious of environmental considerations. In that regard it should be stated that environmentalists have requested that openings in the floodgate structure have "rounded" corners to minimize impact mortality to biota. As well, they would like to see the soffits of flushing gates raised while maintaining an invert at adjacent ground level.

4. Your punctual response to this request has been appreciated. My staff looks forward to working with you to finalize the scope of modeling effort. Questions should be addressed to Mr. Chuck Wener, 617-647-8686.

FOR THE COMMANDER:

Encl
wd

RICHARD D. REARDON
Chief, Engineering Division

CF:
CDRUSACE (CEEC-EH)/Mr. Lockhart
CDRUSACE (CEEC-EH)/Mr. Drummond

CEWES-HE-E (CENED-ED-WQ/15 Dec 88) 1st End W. MARTIN/pfp/601-634-4157
SUBJECT: Model Study - Saugus River Flood Control Project

DA, Waterways Experiment Station, Corps of Engineers, P.O. Box 631,
Vicksburg, MS 39180-0631 JAN 89

FOR: Commander, U.S. Army Engineer Division, New England, ATTN: CENED-ED-WQ,
424 Trapelo Road, Waltham, MA 02254-9149

1. As requested in the basic letter, the enclosed proposal has been prepared to perform a model study of the Saugus River Flood Control Project. The proposal describes a comprehensive evaluation of hydrodynamic, sedimentation, and navigation conditions. In addition, the physical model proposed will allow detailed design of the flood gate structure and provide input to the navigation portion of the study.

2. The study is estimated to cost $1,176,000 and require 25 months to complete. There are undoubtedly areas of the proposed study that could be modified or deleted as a result of further meetings among CEWES, CENED, and HQUSACE personnel.

3. We understand that this study is under consideration for CEWES to perform during FY 90 and 91. In order for us to perform the study, we must be authorized the necessary FTE. To assist us in obtaining necessary FTE allocation, it is requested that you take action to have this work entered into the Headquarters, U.S. Army Corps of Engineers FORCON manpower accounting system when the FY 90 and FY 91 estimates are requested. For this study to be considered by the Programs Division of Civil Works Directorate as justification for the necessary FTE authorization, it must be substantiated and validated by a submission from your office.

4. If you have any questions concerning the proposal, please contact Mr. William Martin at 601-634-4157.

FOR THE COMMANDER AND DIRECTOR:

Encl

ROBERT W. WHALIN, PhD, PE
Technical Director

CF:
HQUSACE CEEC-ED-D (Mr. Lockhart)

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02254-9149

REPLY TO
ATTENTION OF

CENED-ED-WQ 15 December 1988

MEMORANDUM FOR: Commander and Director, Waterways Experiment Station, ATTN: WESHE-E/Mr. William D. Martin, P.O. Box 631, Vicksburg, MS 39180-0631

SUBJECT: Model Study - Saugus River Flood Control Project

1. As discussed between Mr. Charles Wener of NED and Mr. William Martin of your staff, NED requests the following information regarding subject proposed model study for inclusion in the planning feasibility report:

a. Description of model(s) to be used and rationale for selection. Evaluation of both physical and mathematical approaches and explanation for recommending a particular technique.

b. Proposal/scope of model studies with outputs identified.

c. Detailed cost estimate.

d. Schedule for study by major activity - bar chart.

e. Estimate of reasonable contingency factor to be applied to gate costs at this time, considering potential changes which may be made during the design phase when modelling is to be conducted.

2. Background material has been forwarded separately to Mr. Martin. Questions can be directed to Mr. Charles Wener at 617-647-8686.

3. A reply is requested by 27 January 1989.

FOR THE COMMANDER:

RICHARD D. REARDON
Chief, Engineering Division

Copy Furnished:

CDRUSACE (CEEC-ED-D)/Mr. Lockhart

PROPOSAL

SAUGUS RIVER FLOOD CONTROL PROJECT
HYDRAULIC STUDY

Introduction

This proposal responds to a letter dated 15 December 1988 from the New England Division (CENED) in which a detailed scope of study along with time and costs were requested for the Saugus River Flood Control Project. Considerable discussions have occurred between the Waterways Experiment Station (CEWES) and CENED personnel on the general outline of a study required to address the important issues in the construction and operation of the Saugus River Project. This proposal addresses the specific questions of design, construction, and operation of the structure as described in the supportive documents that were forwarded to CEWES along with the 15 December letter.

The proposed study will address the important issues discussed in the supplied documentation. We will model the effects of the project design, construction, and operation on tidal currents, flushing, basin tide levels, sedimentation, and navigation. A comprehensive study is proposed that will cover all hydraulic aspects of the project.

Objectives

The objectives of the study are to provide the most cost effective design configuration for the Saugus River Flood Gates, as defined by the minimum number of gates necessary to provide safe and efficient operation of the structure, and evaluate the structure as to its impacts on:

a. Basin tide levels.

b. Sedimentation/erosion.

c. Water circulation.

d. Changes in current velocities.

e. Navigation.

f. Effects of rising sea levels.

Modeling

Approach

Based on the information supplied by CENED and discussions with the CENED hydraulics and navigation personnel, a hybrid study approach is

Encl 1

suggested. This approach will utilize both numerical and physical models to address the numerous and complex concerns raised by the construction and operation of the proposed flood gates.

A global numerical model of the study area will be developed as shown in Figure 1. This model (actually a suite of models) will use the USACE TABS-2 modeling system. This two-dimensional (2D), depth-averaged model will be used to evaluate global effects on circulation, tide levels, and sedimentation. This model is entirely appropriate for this estuary due to the lack of vertical salinity stratification. The model can be used to screen various scenarios of gate configuration with the objective of minimizing the number of gates required. The global model will also provide boundary conditions for use in the physical model.

The physical model will represent a section of the estuary including the flood gates themselves. This area is shown in Figure 2. This model, which will be constructed such that 1 ft in the model represents 25 ft in the prototype, will provide detailed information needed for the design of the gate structure. The physical model will also provide detailed data on the three-dimensional (3D) flow fields in the vicinity of the structure. These data, in conjunction with the numerical model, will provide input to the navigation study.

The navigation portion of the study will be conducted using the CEWES Ship/Tow Simulator and will evaluate the safety of navigation both during and after construction of the flood gates. The extent of the navigation model will cover an area shown in Figure 3. The primary focus for the vessel simulation study will be the proposed tidal flood gates to be constructed near the entrance to the channel. The design condition increase of currents from 1 knot to 3 knots and constriction of the channel to a 100-ft-wide miter gate section has the potential of causing navigation difficulties for the vessels passing through this channel. A simulation will provide valuable information concerning the effects of these changes on local navigation. Since navigation must be maintained during construction, all of the various construction phases must be included in this study. In the unlikely event that the numerical modeling indicates significant changes in currents in other parts of the navigation channel due to the flood gates or construction work, a reexamination and possible expansion of the simulation test program would be required.

Numerical Model

This entire set of models will be used interactively to develop the most economical gate configuration while minimizing impacts of the structure on the existing conditions. The following project conditions will be modeled according to the construction schedule provided by CENED. Additional plans can be added as deemed necessary by CENED and CEWES.

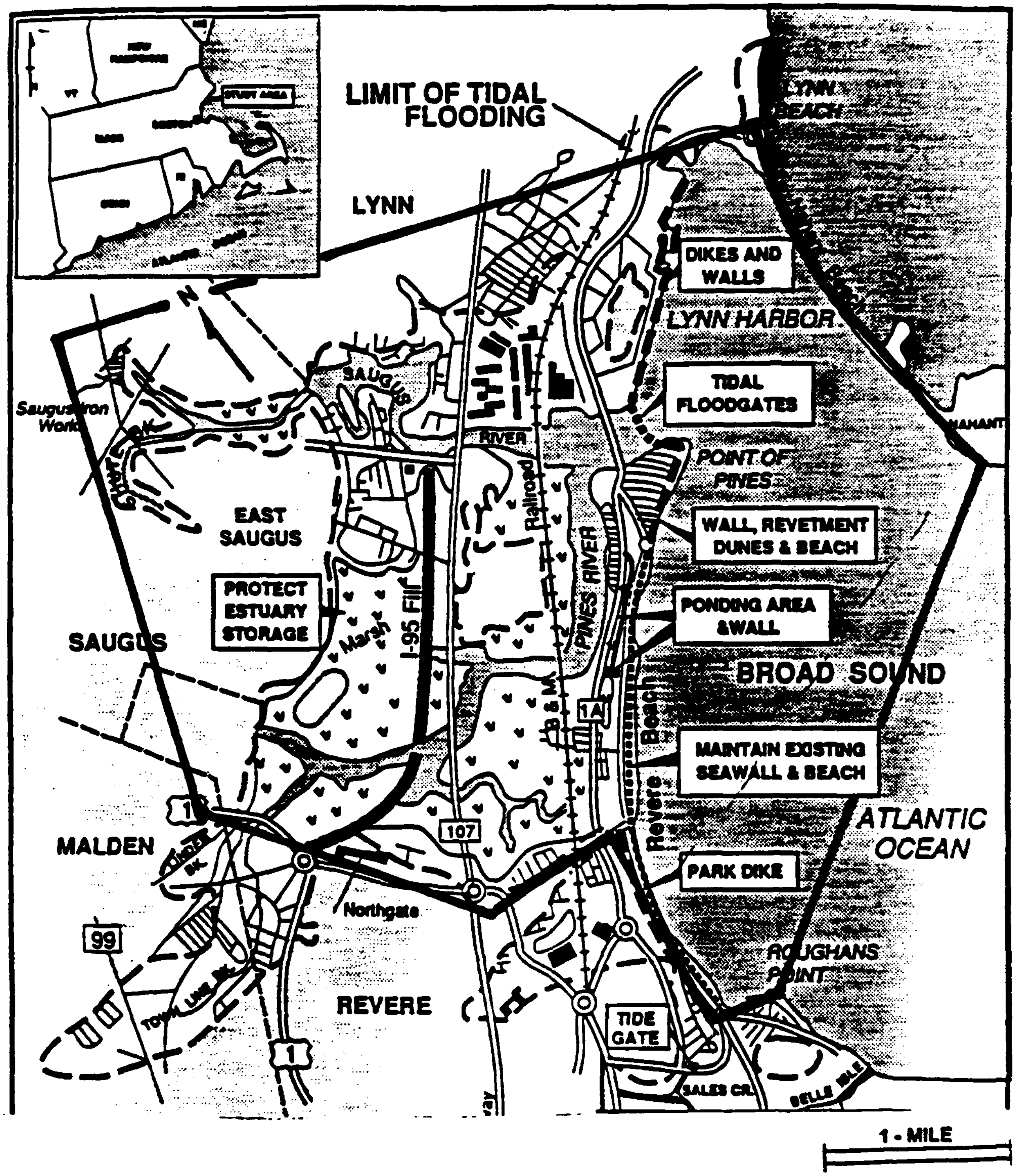

Figure 1. Numerical model limits

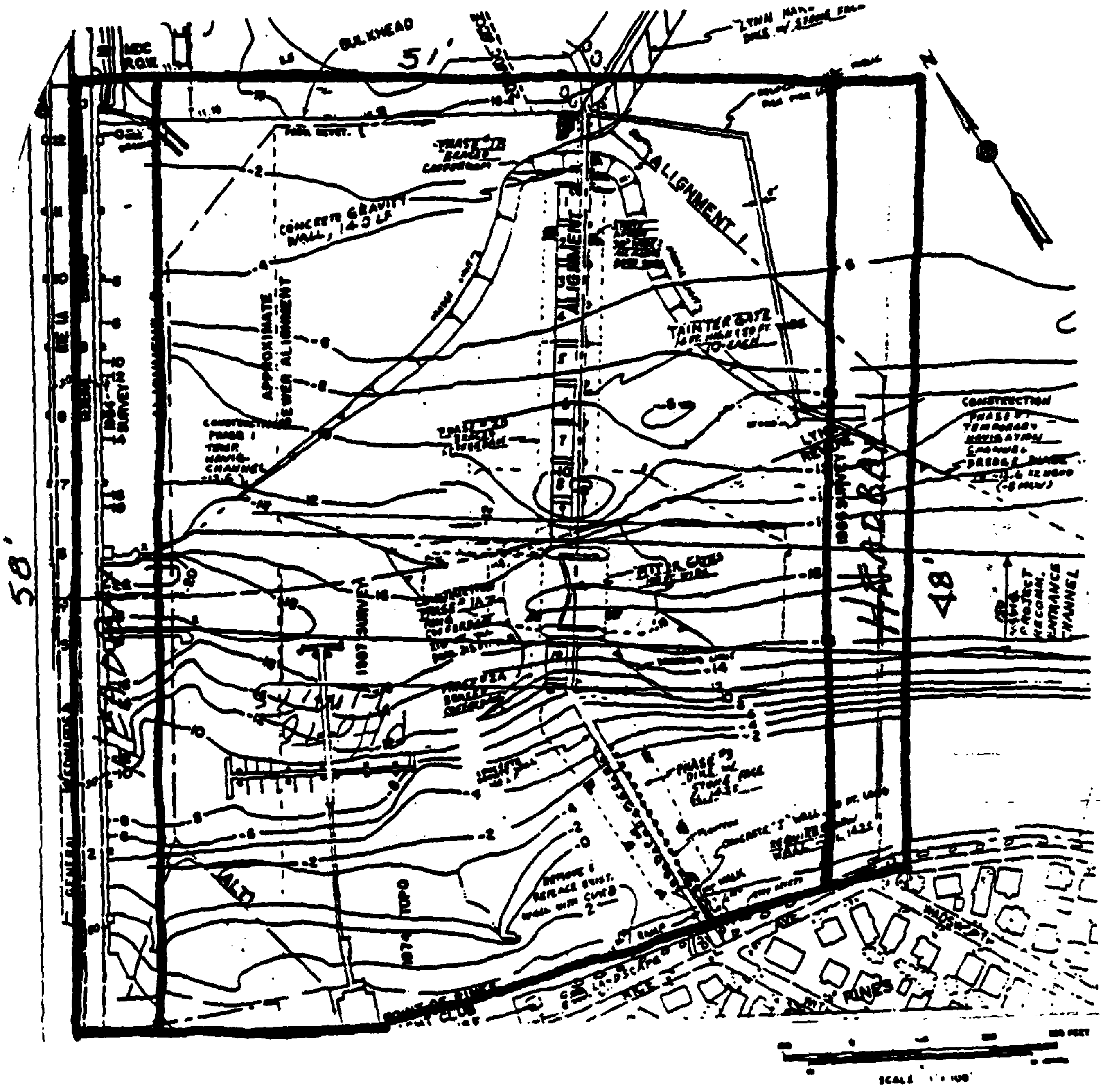

Figure 2. Physical model limits

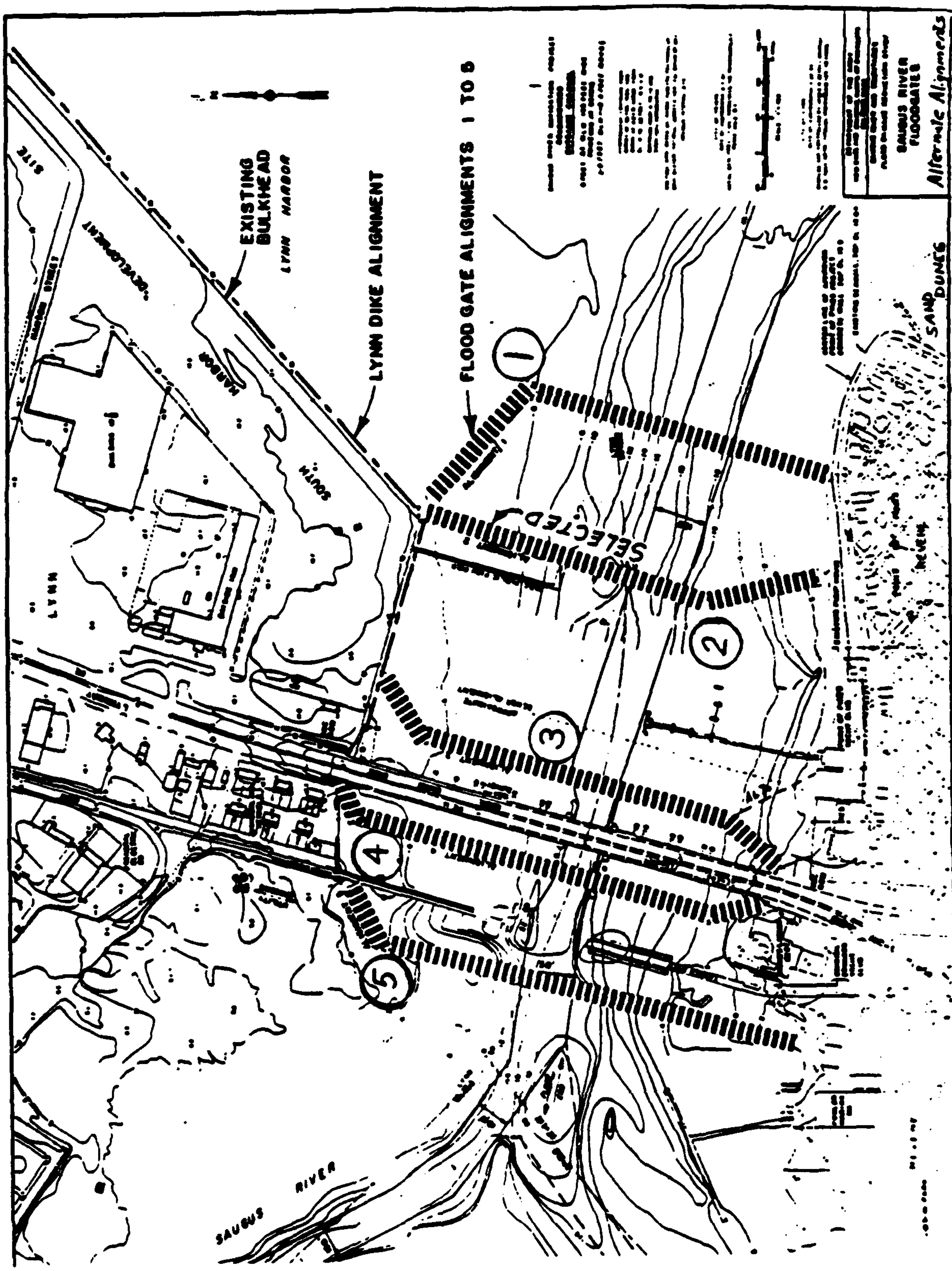

Figure 3. Vessel simulator model area

Base Test - existing geometry

Base Plan Tests
- Plan 1 - original design plan with gates in normal (open) operating position
- Plan 2 - Plan 1 modified to minimize the number of gates for cost reduction

Construction Phase Tests
- Phase 1A - miter gate cofferdam in place with temporary navigation channel
- Phase 1B - remove miter gate cofferdam and install cofferdam at four Lynn Tainter gates
- Phase 2A - remove cofferdam at Lynn Tainter gates and install cofferdam at Revere Tainter gates
- Phase 2B - install cofferdam for remaining Lynn Tainter gates
- Phase 3A - install Revere dike

Boundary conditions for these conditions will be synthesized versions of those collected in the field for verification purposes. The different phases of construction along with the two plans will be screened using this data set which will be representative of a spring tide range. This will determine if additional testing of the phased construction is necessary. Since care was taken in maintaining sufficient area through the entrance in all phases of construction, it is doubtful that any will provide problems and require further testing.

At this point, two plans and all stages of construction with the selected plan will have been tested using the normal spring tide range. The base and chosen plan will then be tested using the maximum astronomical tide along with a normal spring tide range elevated to simulate the effects of sea level rise. The amount of tidal plane elevation increase will be decided by CENED. Depending on the results of the vessel simulator study, it may be necessary to test one or more additional plan configurations.

Once the base and final plan conditions have been modeled, they will be screened using the TABS sediment transport model. Historically, the area has not experienced shoaling problems and it is not likely that the project will cause new problems. However, it is possible that there will be redistribution of shoaling and that will be predicted by the numerical model.

Additional analysis using analytical methods and modeled flows will be conducted to evaluate the depth of local scour near the structure. The proposed 30-ft riprap blanket on both sides of the structure will be evaluated using current CEWES criteria and, if necessary, alternate designs will be provided.

<u>Physical Model</u>

The area that will be reproduced to an undistorted scale of 1:25 is shown in Figure 2. The model will reproduce the structure and the Saugus River for about 400 ft seaward and 600 ft landward. The model will be about

58 ft wide and 51 ft long (about 3,000 sq ft) and will be fixed bed. The structure will be constructed to scale and placed in the model to the proper alignment and elevations.

The purposes of the physical model will be to provide (a) data which will ensure that the barrier control structure would be properly sized to meet the basic requirements of the tidal range and flow conditions, (b) loss coefficient for the gates for use in numerical model verification and testing, (c) aid in riprap design, (d) provide current velocity data in the vicinity of the miter gates for the vessel simulation study and (e) allow evaluation of various operating sequences for the gate closing and opening operations.

Model test data will be collected with the model operating in steady-state flow conditions (both in the ebb and flood directions), with fixed bed and homogeneous flow (fresh water). Sufficient current velocity (maximum ebb and flood) and water surface elevation measurements will be obtained to define local influence of the structure both during and after construction. Each test will involve five to six discharges and head differential conditions from lowest to highest expected. Surface current pattern photographic mosaics will be constructed for both base and plan conditions. The model will provide detailed flow patterns through the structure, and it will provide loss coefficients for use in the numerical model. Depending on the results of the vessel simulator study, it may be necessary to test one or more additional plans.

Navigation Model

The study area consists of a sheltered tidal inlet leading to the navigation channels of the Saugus and Pines Rivers. The navigation channel in the Saugus River is authorized at 150 ft wide and 8 ft deep, mean low water (National Geodetic Vertical Datum). The navigation fleet in the area primarily consists of small commercial and pleasure craft. Lobster boats up to 40 ft long and 5 ft draft comprise the predominant part of the commercial fleet in the Saugus River channel. Pleasure craft up to 50 ft in length also use the local channels. In addition, the largest vessels to use the Saugus River are two types of oil carriers, which deliver oil to a corporate facility 2 to 3 miles inland. At least one of these tankers is a barge/tug configuration while the other is a self-propelled vessel similar to a small tanker. These vessels range up to 245 ft in length, 40 ft wide and 11 ft in draft.

Heretofore, the CEWES ship/tow simulator has been used solely for the simulation of large deep-draft ships or shallow-draft inland river push-tows. Simulation of the small vessels using the Saugus River channel will require additional development activities in order for the existing configuration of the CEWES simulator to be modified to accommodate special requirements. Presently, a research work unit entitled "Small-Craft Coastal Port Design" has been initiated under the Navigation Hydraulics Program in the Civil Works Research and Development Program. Part of this work will concentrate on the data and technology requirements for the simulation of small vessels (less than 100 ft in length). It is evident that this work will be of benefit to projects such as the Saugus River project; however, the exact timing and scope of the effort is still unknown. Therefore, this proposal includes plans for developmental work required for the Saugus River simulation. Specific tasks to be accomplished include towing-tank tests for the development of accurate numerical models of the small design boats. Also, the visual scene generation system presently used in the simulator is not designed for the fast update rate required for realistic simulation of the design vessels; therefore, upgraded computer technology would be needed.

The primary concern for the vessel simulation test program for this channel are the proposed tidal flood gates to be positioned near the entrance of the channel. According to the information provided, in the present condition currents up to 1 knot are common in the Saugus River because of the large tidal range in the area. The addition of the flood gates will increase these currents in the vicinity of the constriction to an estimated 3 knots during certain stages of the tide. Currents of this magnitude can cause difficulty for small vessels; therefore, a simulation study is considered important because it would provide valuable information concerning the effect of this increase on local navigation.

It is our understanding that the exact configuration of the flood gates is still to be decided based on the effect of different alternatives on navigation. In addition, construction time for the project apparently is rather extensive and may cause temporary navigation difficulties. Since safe navigation conditions must be maintained at all times, all construction phases and any design alternatives should be tested in the simulator. Prior to actual testing, a process can be carried out for the purpose of minimizing the number of test scenarios to those most critical.

The navigation study will be initiated by updating the simulator models to represent the class of vessels to be modeled. A trip to the study area to gather input on navigation conditions and record visual scenes for incorporation into the ship simulator will be conducted. The above, along with channel geometry, harbor layout information, output from the TABS-2 hydrodynamic analysis, and physical model results in the vicinity of the miter gates will be compiled to create the data base necessary for the simulation study. The data base will then be validated and tested with local pilot masters or captains from the study area. The selection of these individuals will be coordinated with CENED. The results of the simulation testing will be plotted and evaluated and statistical analysis of vessel control parameters will be performed.

If any design modifications are indicated at this stage, they will be tested in the numerical/physical model and the results again evaluated in the vessel simulator. This procedure will ensure the best possible design that provides safe navigation.

Data Requirements

In order to verify the models described above, a field data collection effort will be necessary. This effort will be conducted as described below.

Approach

A field data collection program will be performed encompassing several tidal cycles over a 1-month period. Continuous information to be obtained throughout this period will include tide levels at seven locations and velocities at three moored meter locations in the study area as indicated in Figure 4. The tide level recorders and moored meters will be installed and left in place to record continuously for the duration of the study period.

After completion of the tide level recorder installations and moored velocity meter deployment, a total of 12 stations will be monitored for collection of velocity data throughout a single tidal cycle. The stations that will be monitored are shown in Figure 4. These velocity stations will be monitored at approximately 30-minute intervals for approximately 13 hours. During this period of data collection at the velocity stations, a limited number of water samples, not to exceed one sample per hour per station, will be obtained for laboratory analysis to determine suspended sediment concentrations. The data to be obtained during this survey will include current speed and direction, as well as the suspended sediment samples. Salinity and temperature will be recorded at stations where the equipment deployed has that capability.

Equipment

All required pressure transducer-type tide level recorders and moored velocity meters will be provided by CEWES. The tidal-cycle data will be obtained from equipment that is deployed from boats located at the specified stations within the study area. The boats and sampling equipment that are to be used will be supplied by CEWES. A total of three vessels, complete with sampling equipment, will be used during the data collection effort. This equipment will include current meters, direction indicators, winches, collapsible equipment booms, sampling water pumps, buoys, anchors, and sample bottles. The seven tide level recorders will be pressure sensor-type ENDECO Model 1029 or equivalent. The moored velocity meters will be the self-contained recording ENDECO Model 174 SSM current meter. Over-the-side velocity meters will be Gurley Model 665 vertical axis cup-type impeller meters.

Procedure

Prior to the 13-hour data collection survey, several days will be needed to install all the tide level recorders at the various locations in the study area. Each boat used in the data collection effort will have a minimum of two personnel from CEWES to operate the equipment and record the data. During the intensive 13-hour data survey, the current speed, current direction and sediment sampling equipment will be deployed simultaneously from each vessel with readings being recorded for the duration of the survey but at intervals of approximately 30 minutes. Each boat will have approximately four velocity stations to monitor. After sampling the last station of the four assigned to a particular boat, it will proceed to the first monitoring station and begin the process again. This sampling procedure will be used throughout the 13-hour period. The depths of water within the rivers and estuary are

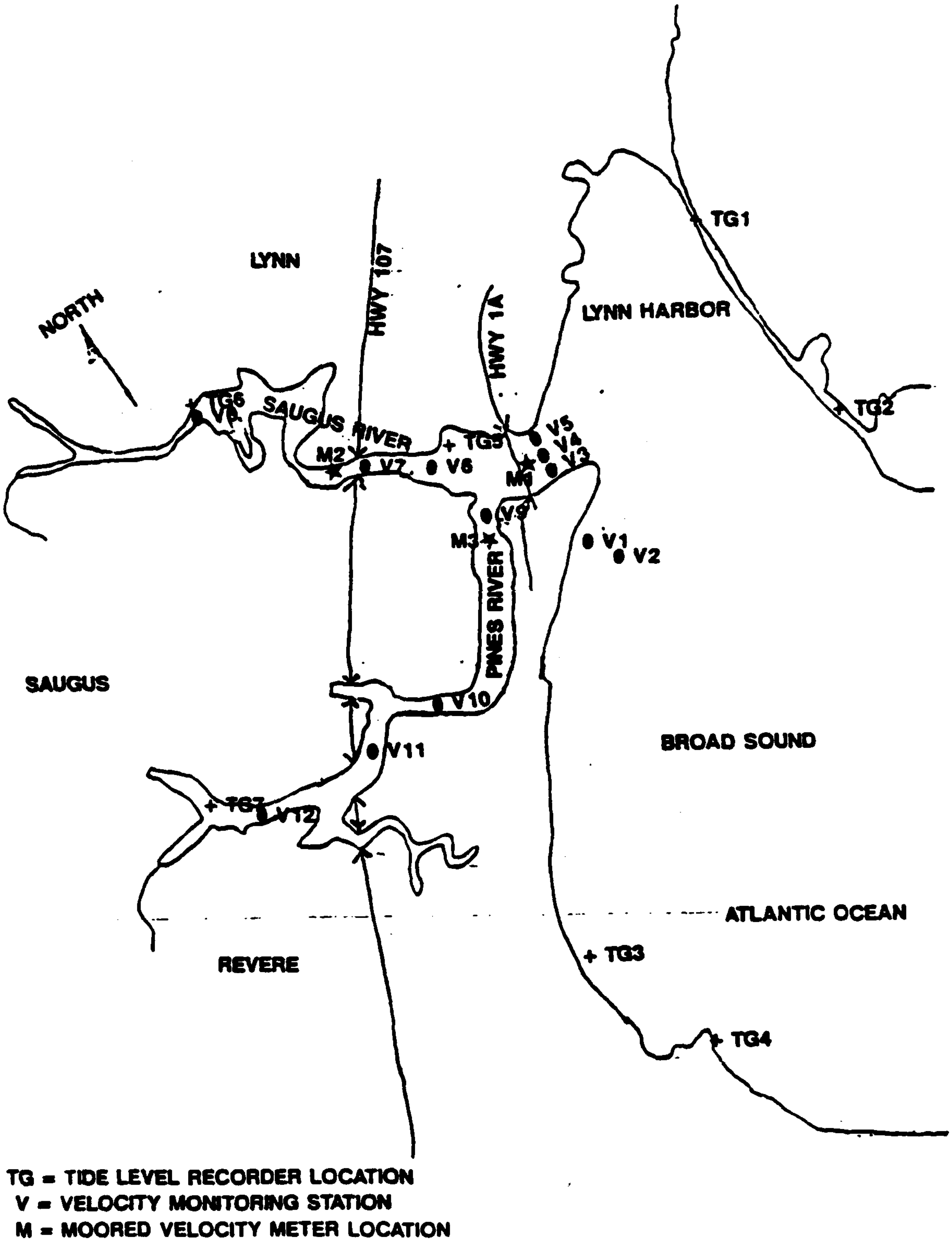

Figure 4. Field data collection stations

expected to be quite varied. To compensate for these variations in depths throughout the study area, at those locations having depths less than 35 ft, measurements will be taken at only three different depths. If depths greater than 35 ft are encountered, measurements will be taken at five depths. The stations to be monitored will be marked by deploying large inflated buoys anchored at the designated locations and from which the vessels will be tethered while data are being collected.

Postponement of the collection due to inclement weather, such as dense fog or extremely rough seas, is a possibility and should such delays occur, additional costs will result and are not reflected in the cost estimate included in this proposal. It should also be noted that although there is no rental charge on equipment deployed for long-term monitoring such as tide level recorders and moored velocity meters, there will be project costs associated with any repair or replacement required due to damage or loss of equipment. The proposed study area is known to be located in a region where a certain type of commercial fishing industry, common only to this area, exists as the livelihood of many of the local inhabitants. The type of deployment necessary for boat mooring lines during the survey and for the moored velocity meters may be disruptive to this enterprise. Possible reparation or litigation costs are not included. The field personnel from CEWES will make every effort possible to prevent this from occurring, but it is requested that CENED coordinate the data collection activities with representatives of the local fishing industry.

Additional data required

In addition to the field data collection described above, information is required about the existing hydrography of the area to be modeled, including the areas beyond the navigation channel, channel and structure design details (usually in the form of drawings) including the construction phase details, navigation charts, aerial photographs and land based maps of developments on the shore area, dredging charts, design vessel or fleet characteristics, and operational information describing navigation conditions. This information will be provided by CENED.

Data reporting

All the data collected in the field will be brought back to CEWES for in-house tabulation and formatting. The information will be made accessible for use as it becomes available. The current velocity data will be transcribed from the field data recording sheets and entered into a computer data file which will tabulate the data. The current velocity data will provide information on the changes in the speed and direction of the currents at each station as they are affected by depth and tides. The collected water samples will be analyzed for suspended sediment concentrations as affected by tide-induced circulation patterns and wind effects. The data from the tide level recorders will be transcribed into a listing of tide level changes with respect to time. The results of the data reduction will be interpreted to identify physical effects of the tide level changes on current velocities and directions.

Cost Estimate

Costs for the various phases of the study are shown below:

Field Survey	$200,000*
TABS-2 Hydrodynamic Modeling	$185,000
TABS-2 Sediment Modeling	$ 40,000
Riprap and Erosion Evaluation	$ 15,000
Physical Model	$336,000
Navigation Model	$400,000
TOTAL	$1,176,000

* Contingencies not included in estimate:
(1) Weather postponement - not to exceed $3,800/day
(2) Equipment loss/repair - not to exceed $10,000/moored velocity meter
- not to exceed $3,500/tide level recorder

Schedule

A detailed task list is provided as encl 1. Enclosure 2 is a PERT chart showing the interdependence of the various tasks. Enclosure 3 is a Gantt chart showing the study activities over time. For reference, the study is shown to start on 1 October 1989. As outlined, the entire study will require 25 months to complete, including publishing of final reports.

Contingency Factors

CENED has specifically requested that CEWES include as a part of this proposal an estimate of reasonable contingency factors to be applied to the gate costs at this time, considering potential changes which may be made during the design phase when the modeling is to be conducted. Such estimates are not typically included in CEWES proposals, but the necessity of such contingencies, prompted by the Local Cooperating Agency agreement, are understood.

It is only appropriate that factors within the scope of the proposed study be included in the contingency estimates. These are discussed below with the rationale for their selection.

Flushing gates. There is a high probability that the number of gates can be reduced by one or two. Some type of debris boom and/or warning system may be desirable to keep debris and small boats from using the flushing gates instead of the navigation channel. Contingency factor: 0 to -10%

Navigation. It is possible that handling of vessels in 3-knot currents through the 100-ft entrance gates will be unsafe. If so, the miter gate dimensions would have to be increased to reduce the velocities or modifications to the construction designs will be required. Contingency factor: +40%

Maintenance Dredging. It is not believed that maintenance dredging will increase significantly beyond that currently experienced. Contingency factor: 0%

Riprap. The 30-ft-wide blanket on both sides of the structure seems too narrow. Based on the selected plan, this will likely need to be increased in areas of high velocities. Contingency factor: +40%

Results

The studies presented above will provide a number of useful results that will materially add to the success of the flood gate design. These include:

a. Optimized number of gates.

b. Assurance of safe navigation conditions, including the critical construction phases.

c. Recommended operation sequence for closing and opening the flood gates.

d. Identification and evaluation of any changes in water circulation patterns or elevations.

e. Identification and evaluation of any changes in sedimentation patterns.

f. Input to and evaluation of design details for the gates and structure.

g. Evaluation of sea level rise on the Saugus estuary.

h. Recommendations as to the extent of local scour near the structure.

i. Riprap design for the plan conditions.

j. Changes in areas inundated behind (landward) of the structure.

Reporting

Preliminary results for each of the four sub-elements of the study, field data collection, numerical modeling, physical modeling, and navigation modeling, will be available within 90 days of completion of the sub-element testing. A total of four final reports will be furnished, each describing the testing, results, and conclusions drawn from each of the four sub-elements of the study. These will be completed within 90 days of completion of sponsor review of the preliminary reports.

Outline
01-25-89 5:00p

Project: SAGUS.PJ
Revision: 20

Heading/Task Resource	Task ID	Pr	Dur	Schd Start	Schd Finish	Allc	Un	Total Hours	Ovr Hours	Task Type	Status	Start Delay	Finish Delay	Float	Free Float
SAGUS.PJ	P1		555	10-02-89<	11-29-91			0	0			0	0	0	0
FIELD DATA	001		120	10-02-89	03-27-90			0	0	ASAP	Scheduled	0	0	435	435
EQUIP LEASE	002		30	10-02-89	11-14-89			0	0	ASAP	Scheduled/Crit.	0	0	0	0
PREPARATION	003		30	11-15-89	12-28-89			0	0	ASAP	Scheduled/Crit.	0	0	0	0
SURVEY	004		10	12-29-89	01-12-90			0	0	ASAP	Scheduled/Crit.	0	0	0	0
CLEAN-UP	005		10	10-02-89	10-16-89			0	0	ASAP	Scheduled	0	0	545	545
SAMPLE ANALYSIS	006		20	01-16-90	02-12-90			0	0	ASAP	Scheduled/Crit.	0	0	0	0
DATA REDUCTION	007		15	02-13-90	03-06-90			0	0	ASAP	Scheduled/Crit.	0	0	0	0
FIELD DATA REP	008		120	10-02-89	03-27-90			0	0	ASAP	Scheduled	0	0	435	435
NUMERICAL MODEL	009		315	10-02-89	12-28-90			0	0	ASAP	Scheduled	0	0	240	240
DATA I D & ORG	010		15	10-02-89	10-23-89			0	0	ASAP	Scheduled	0	0	60	0
MESH DEVELOP	011		30	10-24-89	12-06-89			0	0	ASAP	Scheduled	0	0	60	60
VERIFICATION	012		30	03-07-90	04-17-90			0	0	ASAP	Scheduled/Crit.	0	0	0	0
BASE & PLAN TEST	013		40	04-18-90	06-13-90			0	0	ASAP	Scheduled/Crit.	0	0	0	0
SENSITIVITY TEST	014		40	04-18-90	06-13-90			0	0	ASAP	Scheduled/Crit.	0	0	0	0
ANAL HYDRO	015		20	06-14-90	07-12-90			0	0	ASAP	Scheduled/Crit.	0	0	0	0
SED TESTS	016		15	07-13-90	08-02-90			0	0	ASAP	Scheduled	0	0	240	0
SED ANALYSIS	017		15	08-03-90	08-23-90			0	0	ASAP	Scheduled	0	0	240	0
EROSION EVAL	062		15	08-24-90	09-14-90			0	0	ASAP	Scheduled	0	0	315	315
NUM MOD REP	018		90	08-24-90	12-28-90			0	0	ASAP	Scheduled	0	0	240	240
PHYSICAL MODEL	019		269	10-02-89	10-25-90			0	0	ASAP	Scheduled	0	0	286	286
DESIGN	050		45	10-02-89	12-06-89			0	0	ASAP	Scheduled	0	0	286	0
LAYOUT	020		10	10-02-89	10-16-89			0	0	ASAP	Scheduled	0	0	286	0
TEMPLATES	021		15	10-17-89	11-06-89			0	0	ASAP	Scheduled	0	0	286	0
PLOT TEMPLATES	022		10	11-07-89	11-21-89			0	0	ASAP	Scheduled	0	0	286	0
SHOP DRAWINGS	023		10	11-22-89	12-06-89			0	0	ASAP	Scheduled	0	0	286	0
ANAL FIELD DATA	024		10	10-02-89	10-16-89			0	0	ASAP	Scheduled	0	0	321	321
MODEL CONST	025		90	12-07-89	04-17-90			0	0	ASAP	Scheduled	0	0	286	0
E&CSD-SHOPS	026		90	12-07-89	04-17-90			0	0	ASAP	Scheduled	0	0	286	286
INSTRUMENTATION	028		10	04-18-90	05-01-90			0	0	ASAP	Scheduled	0	0	286	0
INSTALL WLD	029		5	04-18-90	04-24-90			0	0	ASAP	Scheduled	0	0	291	291
INSTALL METERS	030		10	04-18-90	05-01-90			0	0	ASAP	Scheduled	0	0	286	286
PHYS MOD VERIF	031		30	05-02-90	06-13-90			0	0	ASAP	Scheduled	0	0	286	0
LABOR	032		30	05-02-90	06-13-90			0	0	ASAP	Scheduled	0	0	286	286
BASE TESTS	034		15	06-14-90	07-05-90			0	0	ASAP	Scheduled	0	0	286	0
VELOCITIES	035		5	06-14-90	06-20-90			0	0	ASAP	Scheduled	0	0	286	0
ELEVATIONS	036		2	06-14-90	06-15-90			0	0	ASAP	Scheduled	0	0	299	299
PHOTOS	037		5	06-21-90	06-27-90			0	0	ASAP	Scheduled	0	0	286	0
MOSAICS	038		5	06-28-90	07-05-90			0	0	ASAP	Scheduled	0	0	286	286
PLAN TESTS	039		19	07-06-90	08-01-90			0	0	ASAP	Scheduled	0	0	346	346
VELOCITIES	040		5	07-06-90	07-12-90			0	0	ASAP	Scheduled	0	0	286	0
ELEVATIONS	041		2	07-06-90	07-09-90			0	0	ASAP	Scheduled	0	0	358	0
RIPRAP DESIGN	052		5	07-10-90	07-16-90			0	0	ASAP	Scheduled	0	0	358	358
PHOTOS	042		5	07-13-90	07-19-90			0	0	ASAP	Scheduled	0	0	286	0
MOSAICS	043		5	07-20-90	07-26-90			0	0	ASAP	Scheduled	0	0	286	0
ANALYZE RESULTS	044		4	07-27-90	08-01-90			0	0	ASAP	Scheduled	0	0	286	0
PHY MOD REP	051		60	08-02-90	10-25-90			0	0	ASAP	Scheduled	0	0	286	286
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UPDATE MODEL	054		180	10-02-89	06-20-90			0	0	ASAP	Scheduled	0	0	375	375
DESIGN & RECON	055		20	10-02-89	10-30-89			0	0	ASAP	Scheduled	0	0	525	375

Encl 1 to Encl

Heading/Task Resource	Task ID	Early Start	Early Finish	Late Start	Late Finish
SAGUS.PJ	P1	10-01-89<	11-29-91	10-02-89	11-29-91
FIELD DATA	001	10-01-89	03-27-90	10-02-89	11-29-91
EQUIP LEASE	002	10-01-89	11-14-89	10-02-89	11-14-89
PREPARATION	003	11-14-89	12-28-89	11-15-89	12-28-89
SURVEY	004	12-28-89	01-12-90	12-29-89	01-12-90
CLEAN-UP	005	10-01-89	10-16-89	11-18-91	11-29-91
SAMPLE ANALYSIS	006	01-12-90	02-12-90	01-16-90	02-12-90
DATA REDUCTION	007	02-12-90	03-06-90	02-13-90	03-06-90
FIELD DATA REP	008	10-01-89	03-27-90	06-17-91	11-29-91
NUMERICAL MODEL	009	10-01-89	12-28-90	12-29-89	11-29-91
DATA I D & ORG	010	10-01-89	10-23-89	12-29-89	01-22-90
MESH DEVELOP	011	10-23-89	12-06-89	01-23-90	03-06-90
VERIFICATION	012	03-06-90	04-17-90	03-07-90	04-17-90
BASE & PLAN TEST	013	04-17-90	06-13-90	04-18-90	06-13-90
SENSITIVITY TEST	014	04-17-90	06-13-90	04-18-90	06-13-90
ANAL HYDRO	015	06-13-90	07-12-90	06-14-90	07-12-90
SED TESTS	016	07-12-90	08-02-90	06-17-91	07-05-91
SED ANALYSIS	017	08-02-90	08-23-90	07-08-91	07-26-91
EROSION EVAL	062	08-23-90	09-14-90	11-11-91	11-29-91
NUM MOD REP	018	08-23-90	12-28-90	07-29-91	11-29-91
PHYSICAL MODEL	019	10-01-89	10-25-90	11-20-90	11-29-91
DESIGN	050	10-01-89	12-06-89	11-20-90	01-21-91
LAYOUT	020	10-01-89	10-16-89	11-20-90	12-03-90
TEMPLATES	021	10-16-89	11-06-89	12-04-90	12-24-90
PLOT TEMPLATES	022	11-06-89	11-21-89	12-25-90	01-07-91
SHOP DRAWINGS	023	11-21-89	12-06-89	01-08-91	01-21-91
ANAL FIELD DATA	024	10-01-89	10-16-89	01-08-91	01-21-91
MODEL CONST	025	12-06-89	04-17-90	01-22-91	05-27-91
E&CSD-SHOPS	026	12-06-89	04-17-90	01-22-91	05-27-91
INSTRUMENTATION	028	04-17-90	05-01-90	05-28-91	06-10-91
INSTALL WLD	029	04-17-90	04-24-90	06-04-91	06-10-91
INSTALL METERS	030	04-17-90	05-01-90	05-28-91	06-10-91
PHYS MOD VERIF	031	05-01-90	06-13-90	06-11-91	07-22-91
LABOR	032	05-01-90	06-13-90	06-11-91	07-22-91
BASE TESTS	034	06-13-90	07-05-90	07-23-91	08-12-91
VELOCITIES	035	06-13-90	06-20-90	07-23-91	07-29-91
ELEVATIONS	036	06-13-90	06-15-90	08-09-91	08-12-91
PHOTOS	037	06-20-90	06-27-90	07-30-91	08-05-91
MOSAICS	038	06-27-90	07-05-90	08-06-91	08-12-91
PLAN TESTS	039	07-05-90	08-01-90	08-13-91	11-29-91
VELOCITIES	040	07-05-90	07-12-90	08-13-91	08-19-91
ELEVATIONS	041	07-05-90	07-09-90	11-21-91	11-22-91
RIPRAP DESIGN	052	07-09-90	07-16-90	11-25-91	11-29-91
PHOTOS	042	07-12-90	07-19-90	08-20-91	08-26-91
MOSAICS	043	07-19-90	07-26-90	08-27-91	09-02-91
ANALYZE RESULTS	044	07-26-90	08-01-90	09-03-91	09-06-91
PHY MOD REP	051	08-01-90	10-25-90	09-09-91	11-29-91
NAVIGATION MODEL	053	10-01-89	11-29-91	07-13-90	11-29-91
UPDATE MODEL	054	10-01-89	06-20-90	03-25-91	11-29-91
DESIGN & RECON	055	10-01-89	10-30-89	10-21-91	11-15-91

Outline
01-25-89 5:00p

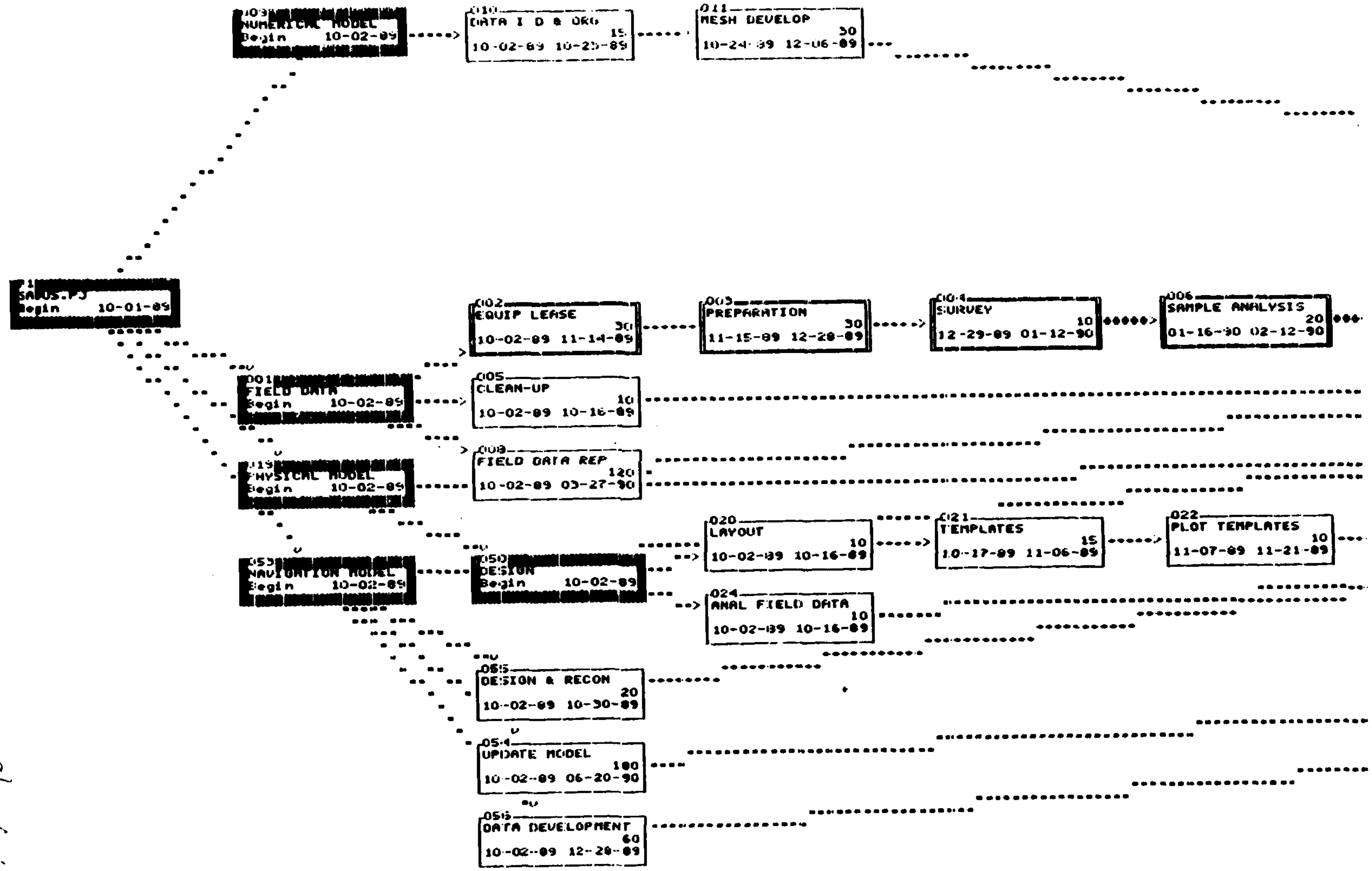
Across: 1 Down: 2
Project: SAGUS.PJ
Revision: 20

Heading/Task Resource	Task ID	Pr	Dur	Schd Start	Schd Finish	Allc	Un	Total Hours	Ovr Hours	Task Type	Status	Start Delay	Finish Delay	Float	Free Float
DATA DEVELOPMENT	056		60	10-02-89	12-28-89			0	0	ASAP	Scheduled	0	0	495	495
TESTING	057		40	07-13-90	09-07-90			0	0	ASAP	Scheduled/Crit.	0	0	0	0
ANALYSIS	058		160	09-10-90	04-19-91			0	0	ASAP	Scheduled/Crit.	0	0	0	0
PRELIM RESULTS	059		10	04-22-91	05-03-91			0	0	ASAP	Scheduled	0	0	150	150
DRAFT REPORT	060		60	04-22-91	07-12-91			0	0	ASAP	Scheduled/Crit.	0	0	0	0
NAV REPORT	061		100	07-15-91	11-29-91			0	0	ASAP	Scheduled/Crit.	0	0	0	0

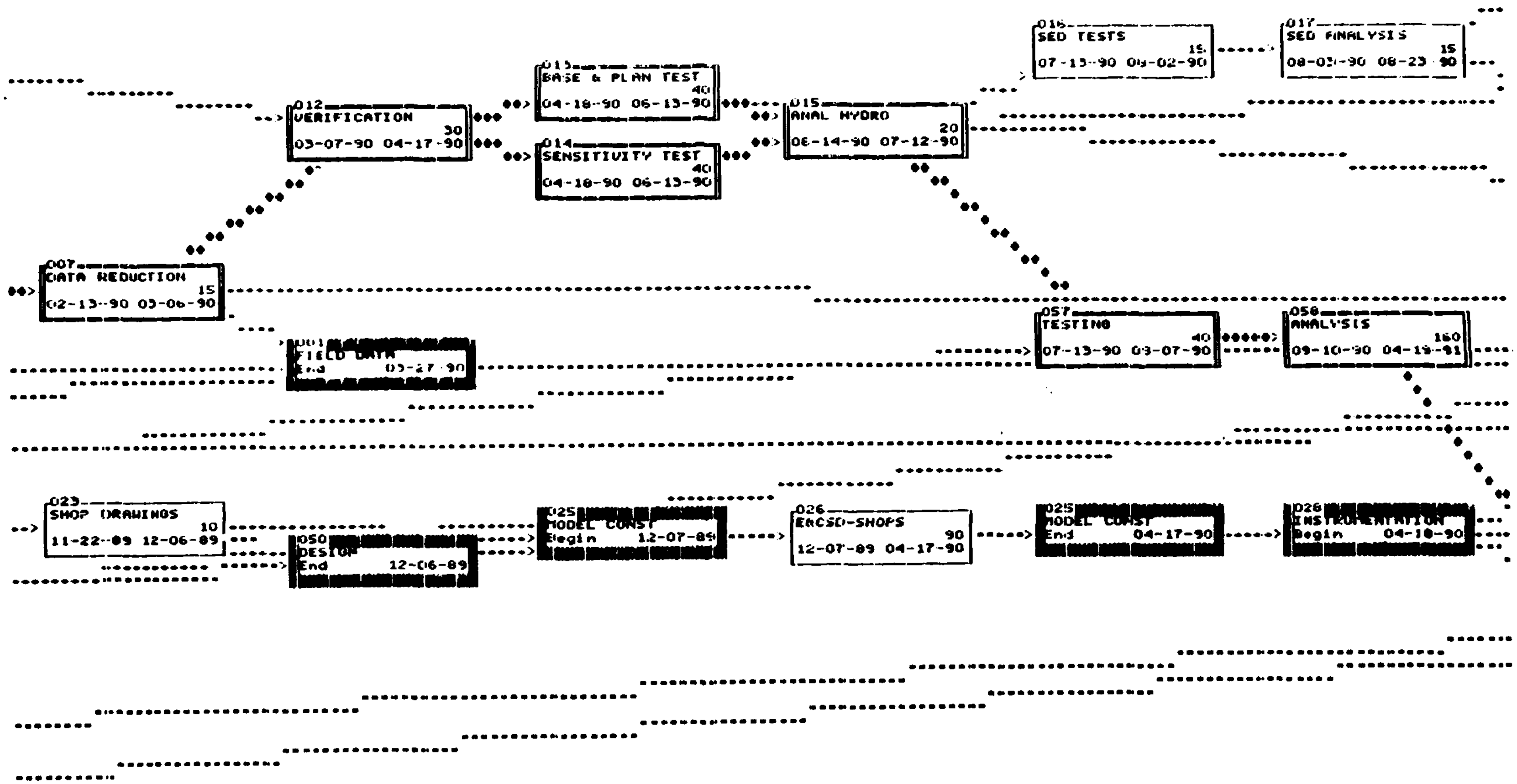
Heading/Task Resource	Early Start	Early Finish	Late Start	Late Finish
DATA DEVELOPMENT	10-01-89	12-28-89	09-09-91	11-29-91
TESTING	07-12-90	09-07-90	07-13-90	09-07-90
ANALYSIS	09-07-90	04-19-91	09-10-90	04-19-91
PRELIM RESULTS	04-19-91	05-03-91	11-18-91	11-29-91
DRAFT REPORT	04-19-91	07-12-91	04-22-91	07-12-91
NAV REPORT	07-12-91	11-29-91	07-15-91	11-29-91

PERT Chart
01-26-89 9:20a

Project: SAGUS.PJ
Revision: 20



---- Link ●●●● Critical Link ---- WBS Link Task Critical Task



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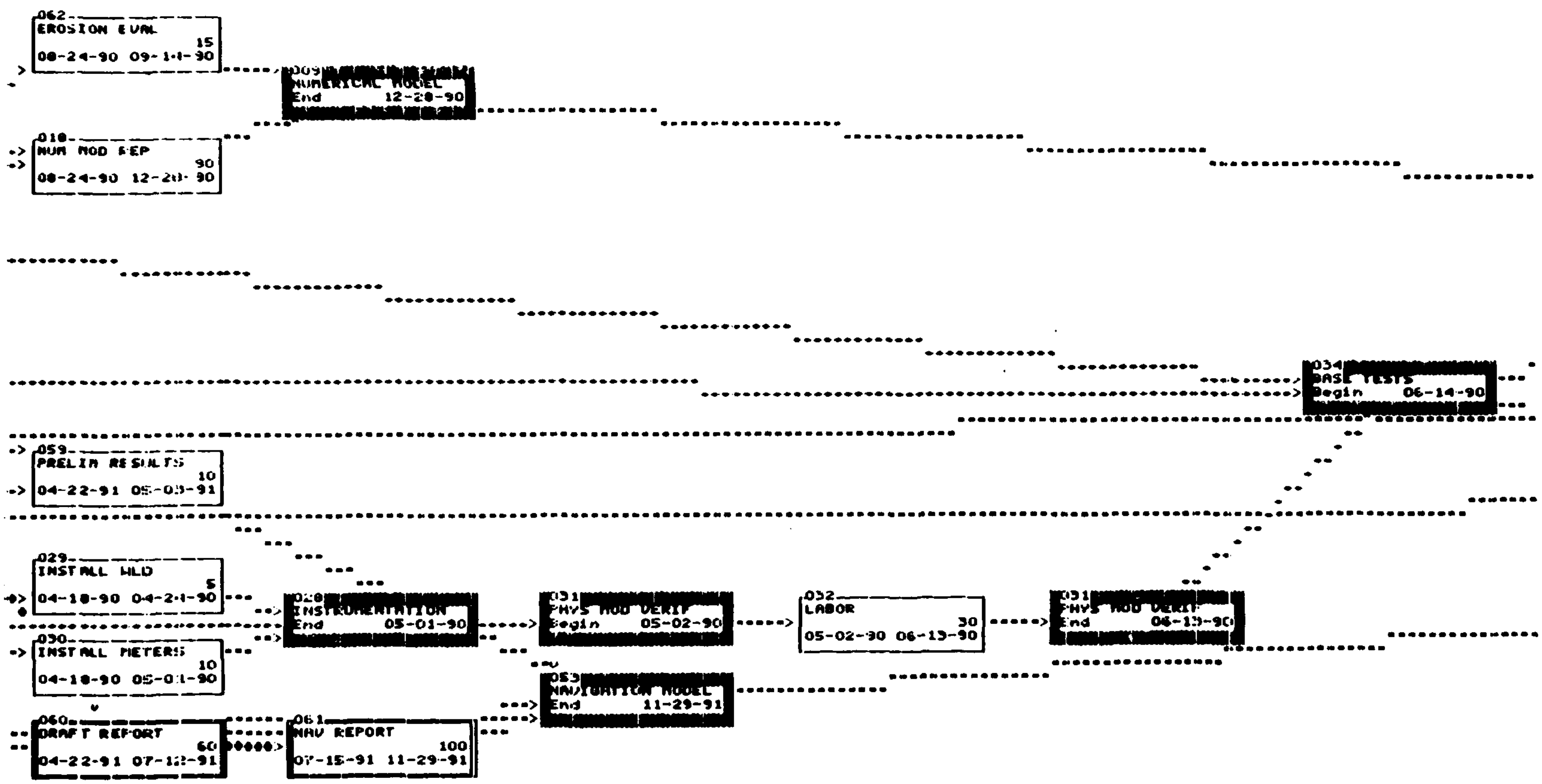
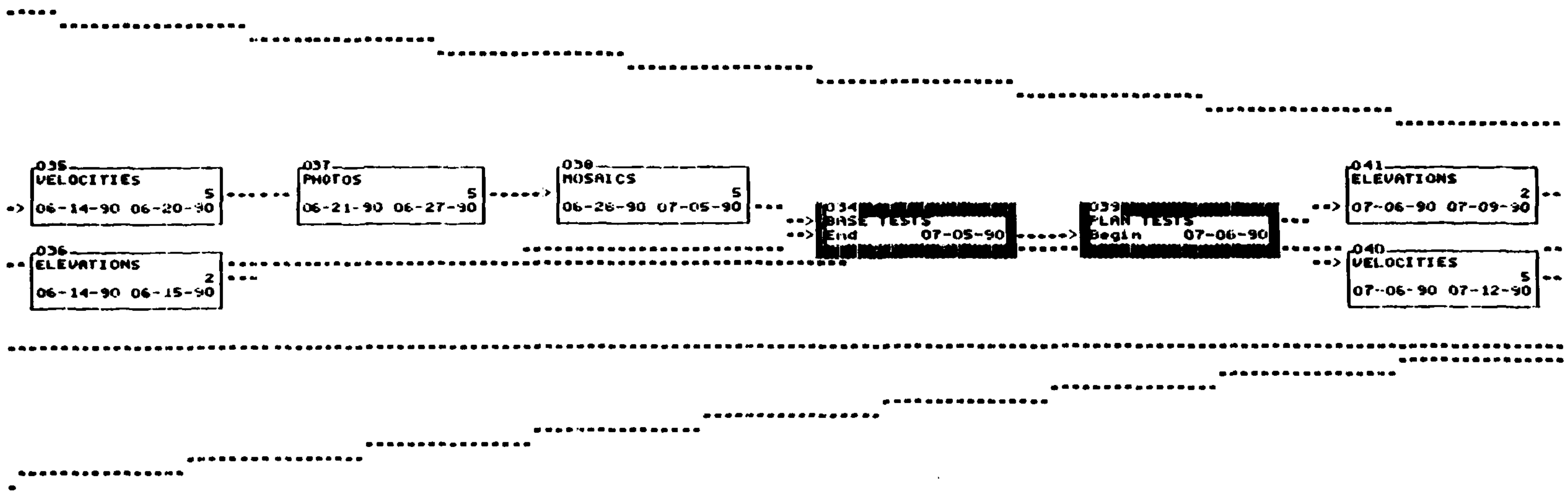
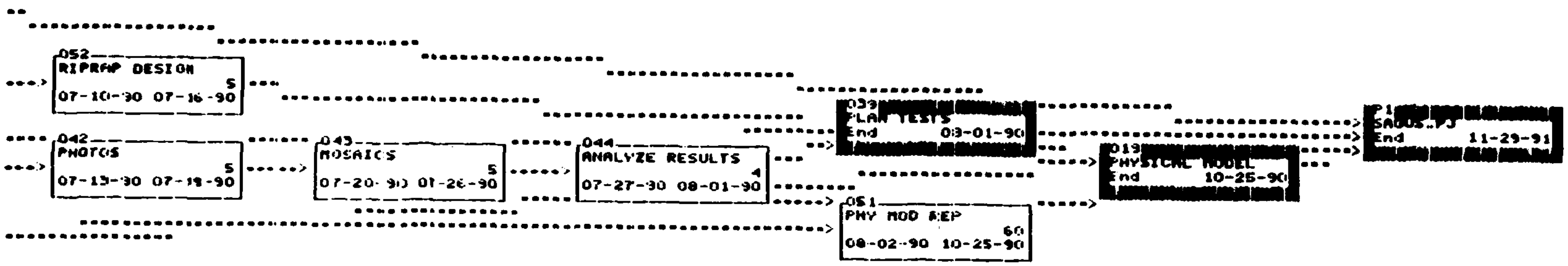
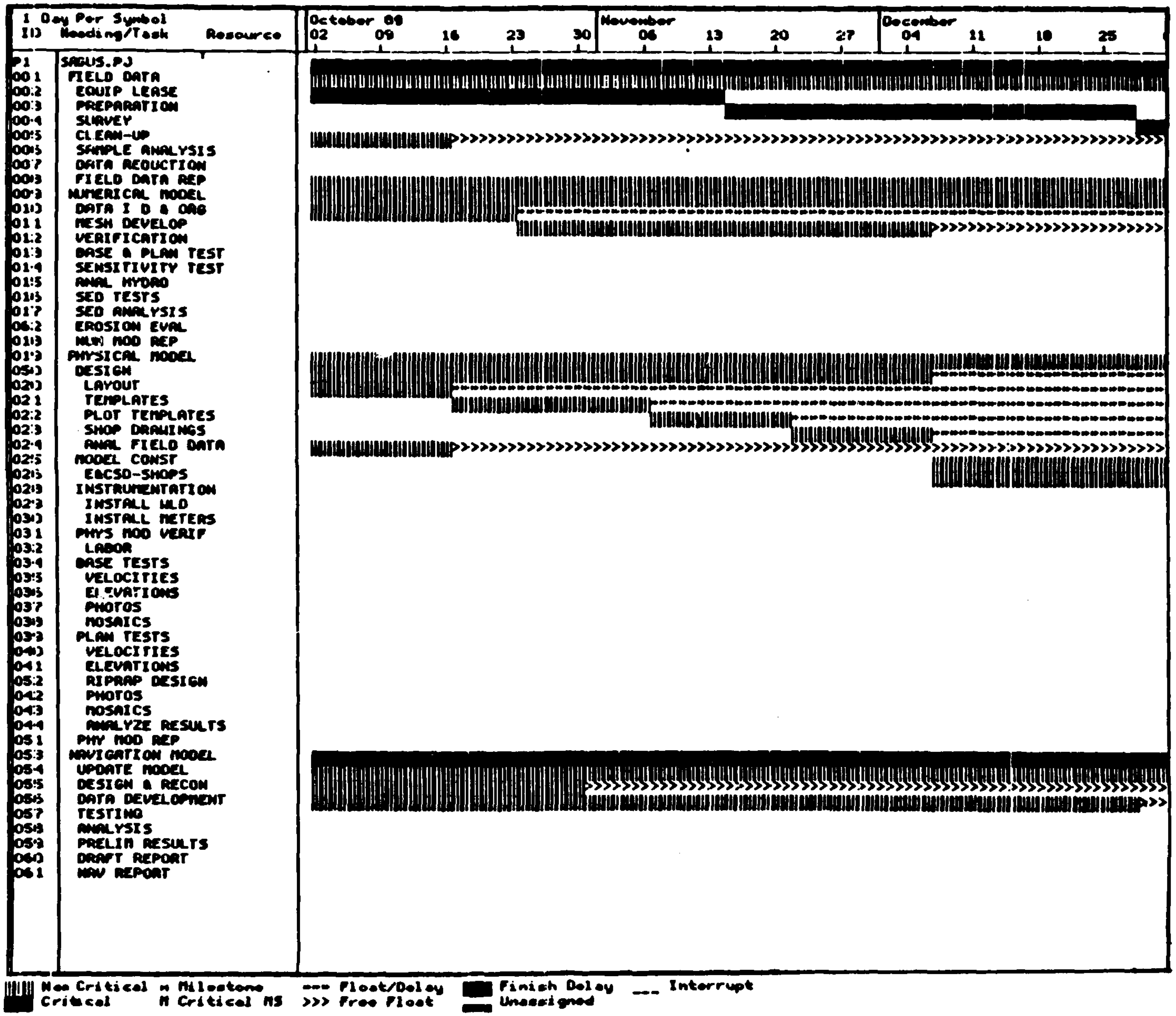
02-13-90 03-06-90
012 VERIFICATION 30
03-07-90 04-17-90
013 BASE & PLAN TEST 40
04-18-90 06-13-90
014 SENSITIVITY TEST 40
04-18-90 06-13-90
015 ANAL HYDRO 20
06-14-90 07-12-90
016 SED TESTS 15
07-13-90 08-02-90
017 SED ANALYSIS 15
08-03-90 08-23-90
FIELD DATA
End 03-27-90
057 TESTING 40
07-13-90 09-07-90
058 ANALYSIS 160
09-10-90 04-19-91
023 SHOP DRAWINGS 10
11-22-89 12-06-89
050 DESIGN
End 12-06-89
025 MODEL CONST
Begin 12-07-89
026 ERCSD-SHOPS 90
12-07-89 04-17-90
025 MODEL CONST
End 04-17-90
026 INSTRUMENTATION
Begin 04-18-90


062
EROSION EVAL
15
08-24-90 09-14-90
009
NUMERICAL MODEL
End 12-20-90
018
NUM MOD PREP
90
08-24-90 12-20-90
034
BASE TESTS
Begin 06-14-90
059
PRELIM RESULTS
10
04-22-91 05-03-91
029
INSTALL WLD
5
04-18-90 04-24-90
028
INSTRUMENTATION
End 05-01-90
031
PHYS MOD VERIF
Begin 05-02-90
032
LABOR
30
05-02-90 06-13-90
031
PHYS MOD VERIF
End 06-13-90
030
INSTALL METERS
10
04-18-90 05-01-90
053
NAVIGATION MODEL
End 11-29-91
060
DRAFT REPORT
60
04-22-91 07-12-91
061
NAV REPORT
100
07-15-91 11-29-91


035
VELOCITIES
5
06-14-90 06-20-90
036
ELEVATIONS
2
06-14-90 06-15-90
037
PHOTOS
5
06-21-90 06-27-90
038
MOSAICS
5
06-28-90 07-05-90
034
BASE TESTS
End 07-05-90
039
PLAN TESTS
Begin 07-06-90
041
ELEVATIONS
2
07-06-90 07-09-90
040
VELOCITIES
5
07-06-90 07-12-90

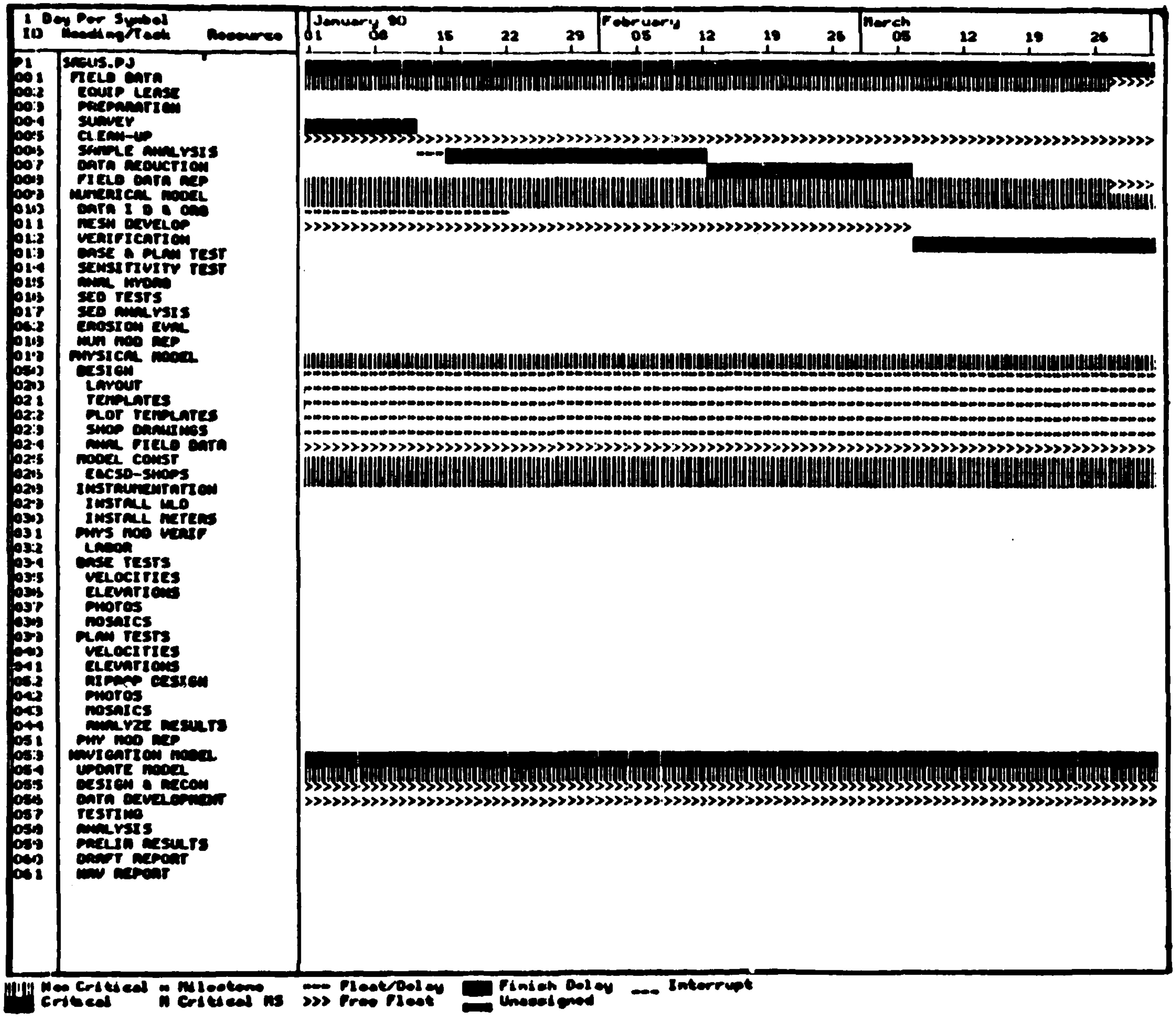

052
RIPRAP DESIGN
5
07-10-90 07-16-90
042
PHOTOS
5
07-13-90 07-19-90
043
MOSAICS
5
07-20-90 07-26-90
044
ANALYZE RESULTS
4
07-27-90 08-01-90
039
PLAN TESTS
End 08-01-90
051
PHY MOD REP
60
08-02-90 10-25-90
013
PHYSICAL MODEL
End 10-25-90
P1
SAOUS.PJ
End 11-29-91

Task Gantt
01-26-89 9:22a

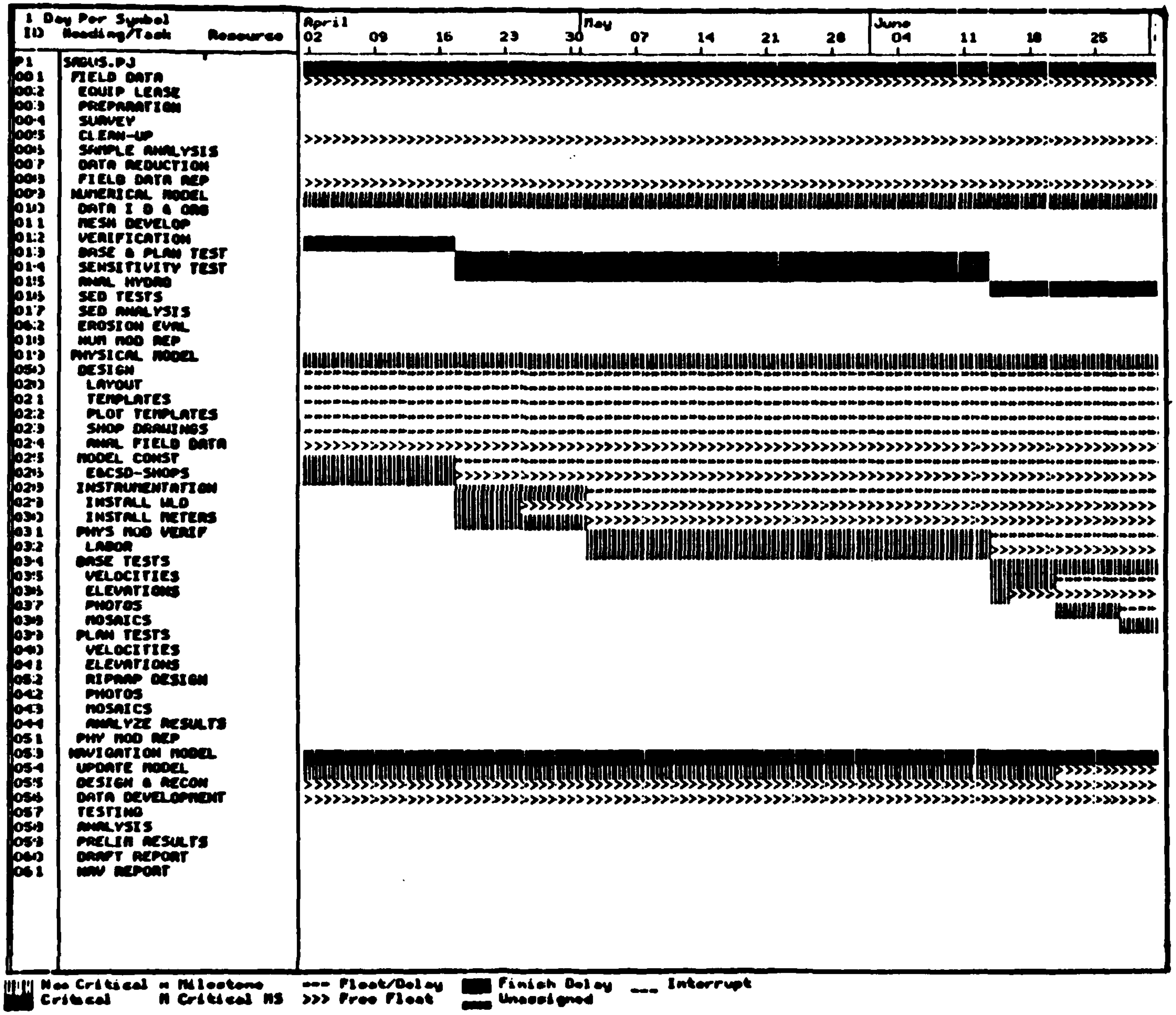


Encl 3 of 3

Task Gantt
01-26-90 9:22a



Task Gantt
01-26-89 9:22a



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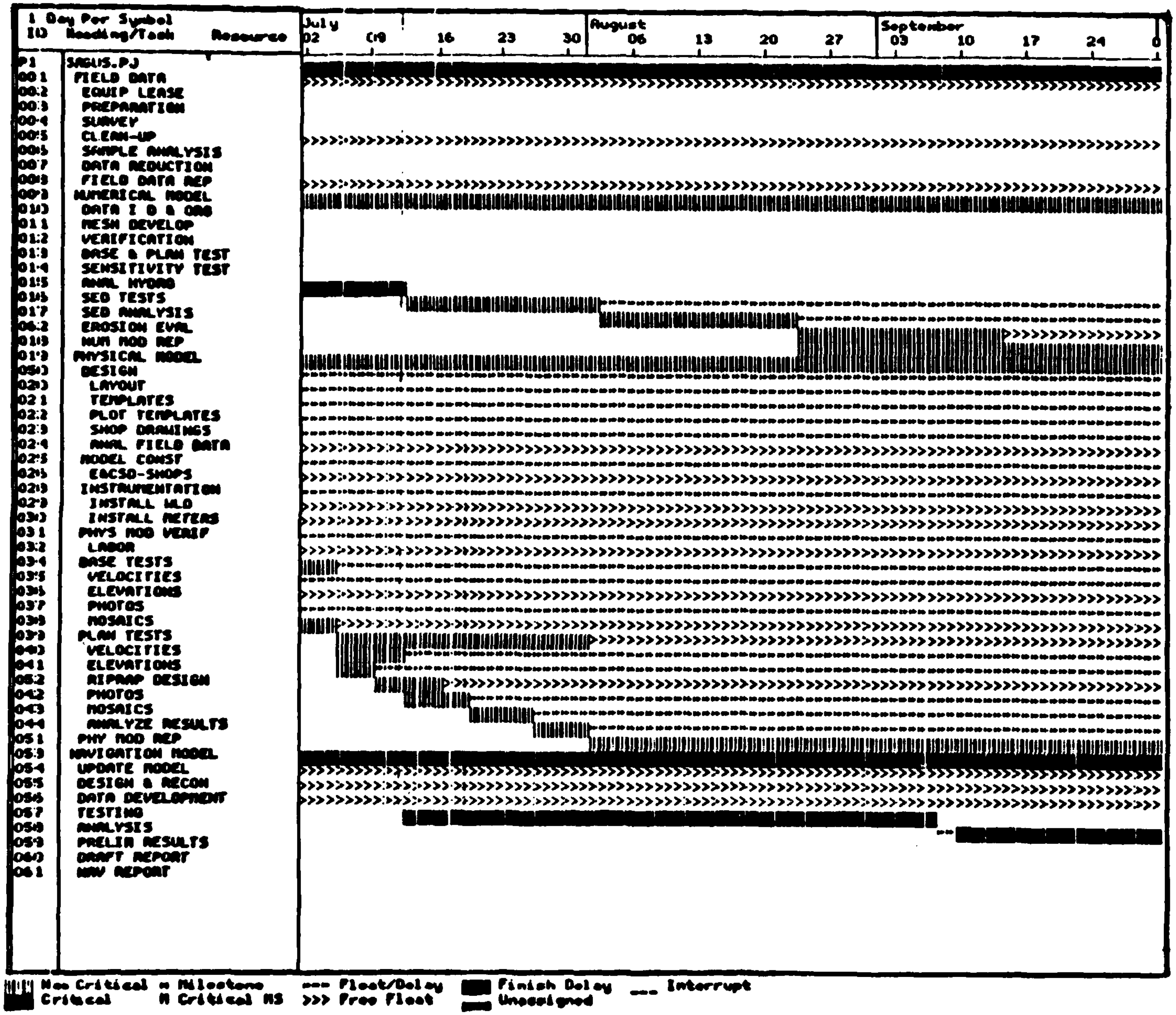
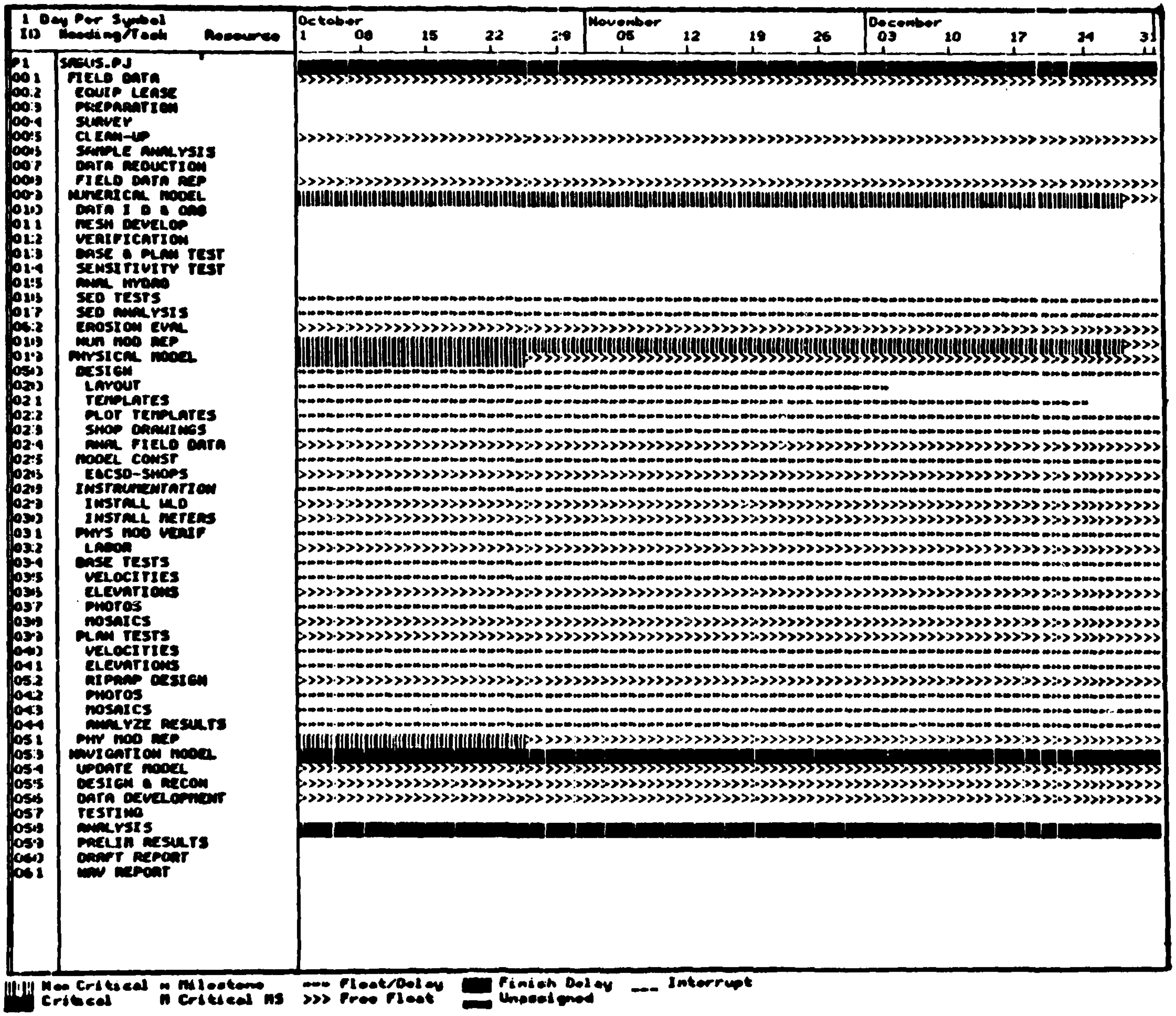
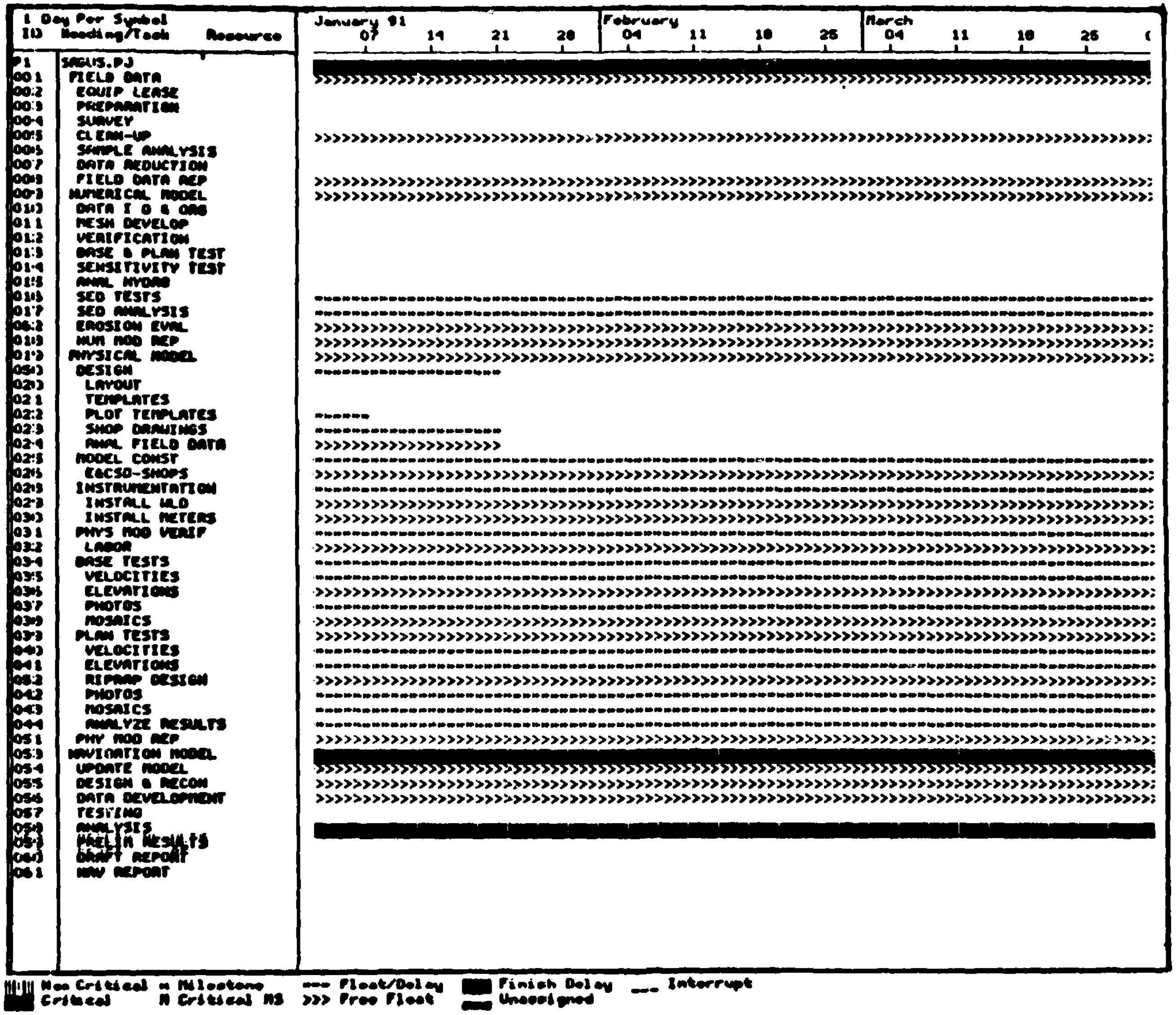
01-26-89 9:22a



Task Gantt
01-26-89 9:22a



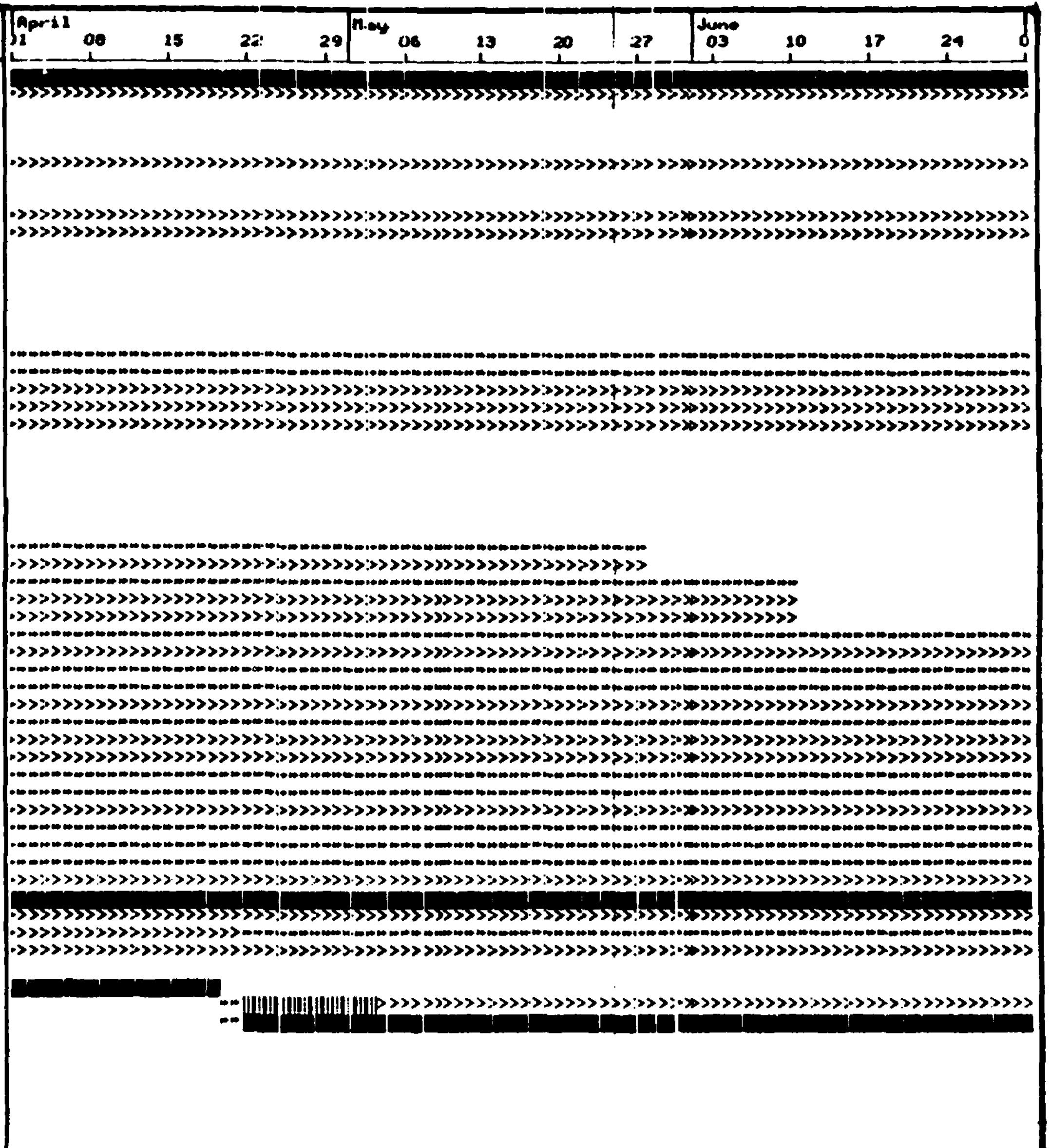
Task Gantt
01-26-89 9:22a



Task Gantt
01-26-89 9:22a

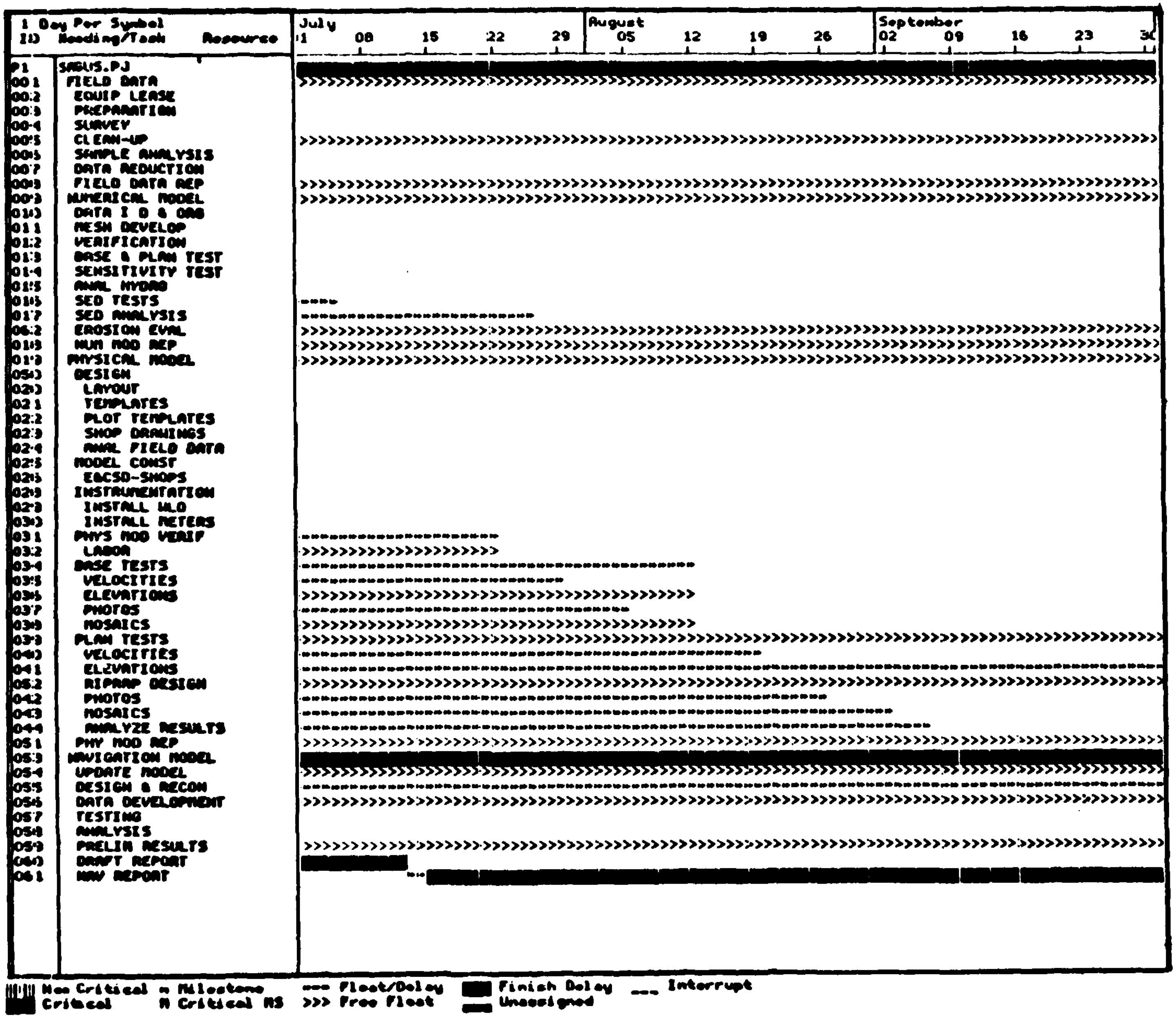
1 Day Per Symbol

ID	Heading/Task	Resource
P1	SAGUS.PJ	
001	FIELD DATA	
002	EQUIP LEASE	
003	PREPARATION	
004	SURVEY	
005	CLEAN-UP	
006	SAMPLE ANALYSIS	
007	DATA REDUCTION	
008	FIELD DATA REP	
009	NUMERICAL MODEL	
010	DATA I O & ORG	
011	MESH DEVELOP	
012	VERIFICATION	
013	BASE & PLAN TEST	
014	SENSITIVITY TEST	
015	ANAL HYDRO	
016	SED TESTS	
017	SED ANALYSIS	
062	EROSION EVAL	
018	NUM MOD REP	
019	PHYSICAL MODEL	
050	DESIGN	
020	LAYOUT	
021	TEMPLATES	
022	PLOT TEMPLATES	
023	SHOP DRAWINGS	
024	ANAL FIELD DATA	
025	MODEL CONST	
026	E&CSO-SHOPS	
029	INSTRUMENTATION	
028	INSTALL WLO	
030	INSTALL METERS	
031	PHYS MOD VERIF	
032	LABOR	
034	BASE TESTS	
035	VELOCITIES	
036	ELEVATIONS	
037	PHOTOS	
038	MOSAICS	
039	PLAN TESTS	
040	VELOCITIES	
041	ELEVATIONS	
052	RIPRAP DESIGN	
042	PHOTOS	
043	MOSAICS	
044	ANALYZE RESULTS	
051	PHY MOD REP	
053	NAVIGATION MODEL	
054	UPDATE MODEL	
055	DESIGN & RECON	
056	DATA DEVELOPMENT	
057	TESTING	
058	ANALYSIS	
059	PRELIM RESULTS	
060	DRAFT REPORT	
061	NAV REPORT	



Non Critical = Milestone --- Float/Delay Finish Delay ___ Interrupt
Critical M Critical MS >>> Free Float Unassigned

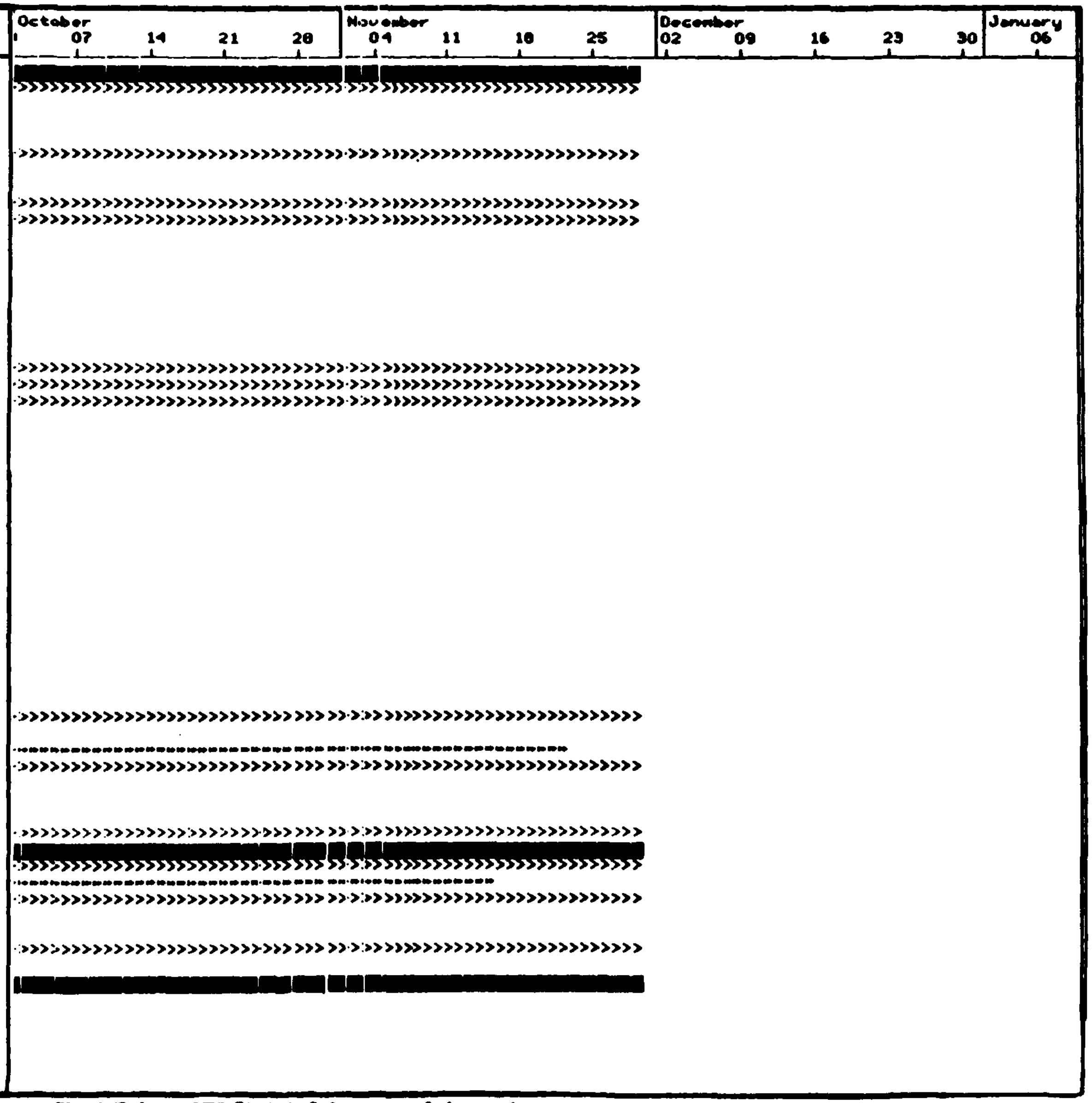
Task Gantt
01-26-89 9:22a



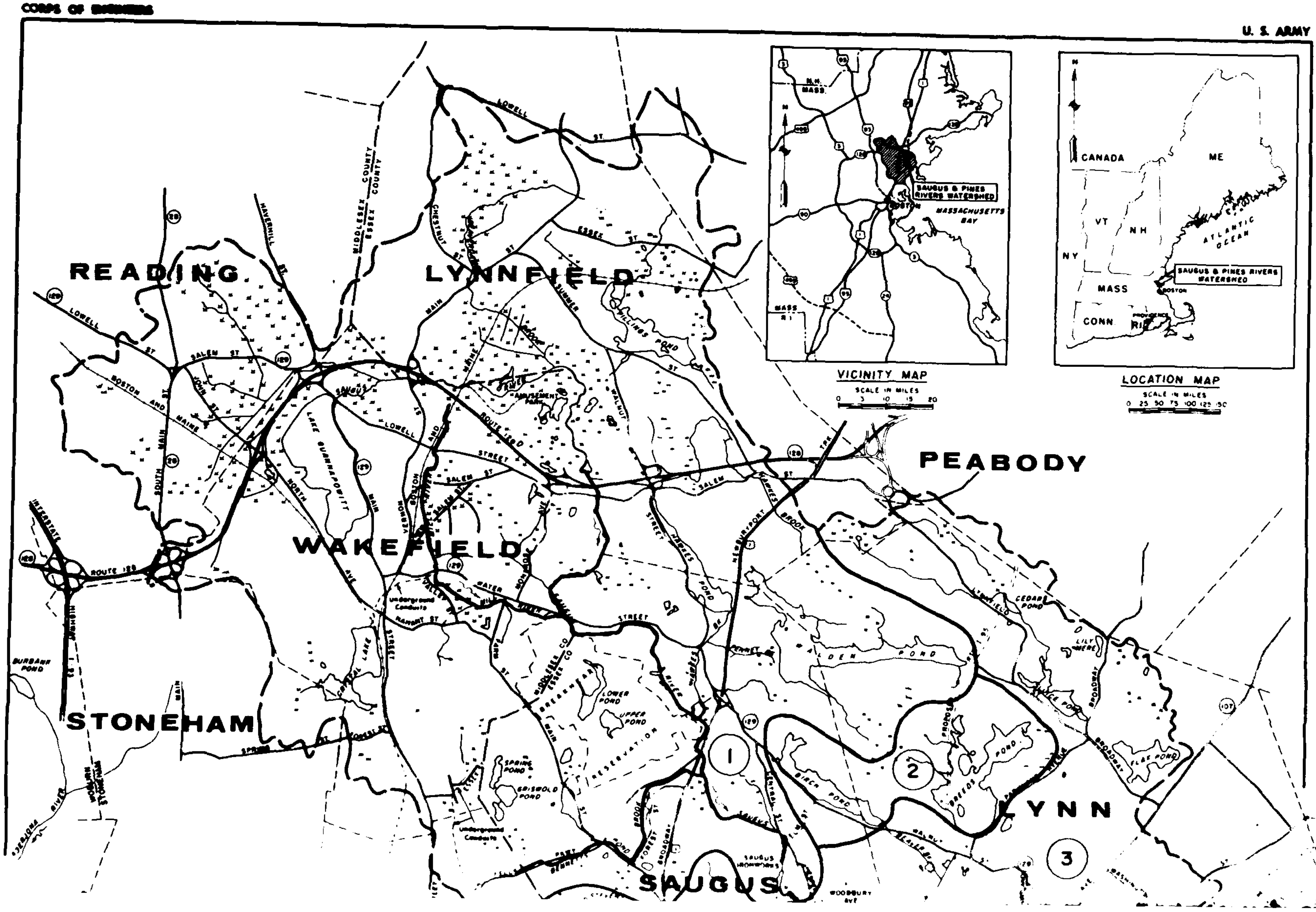
Task Gantt
01-26-89 9:22a

Project: SAGUS.PJ
Revision: 20

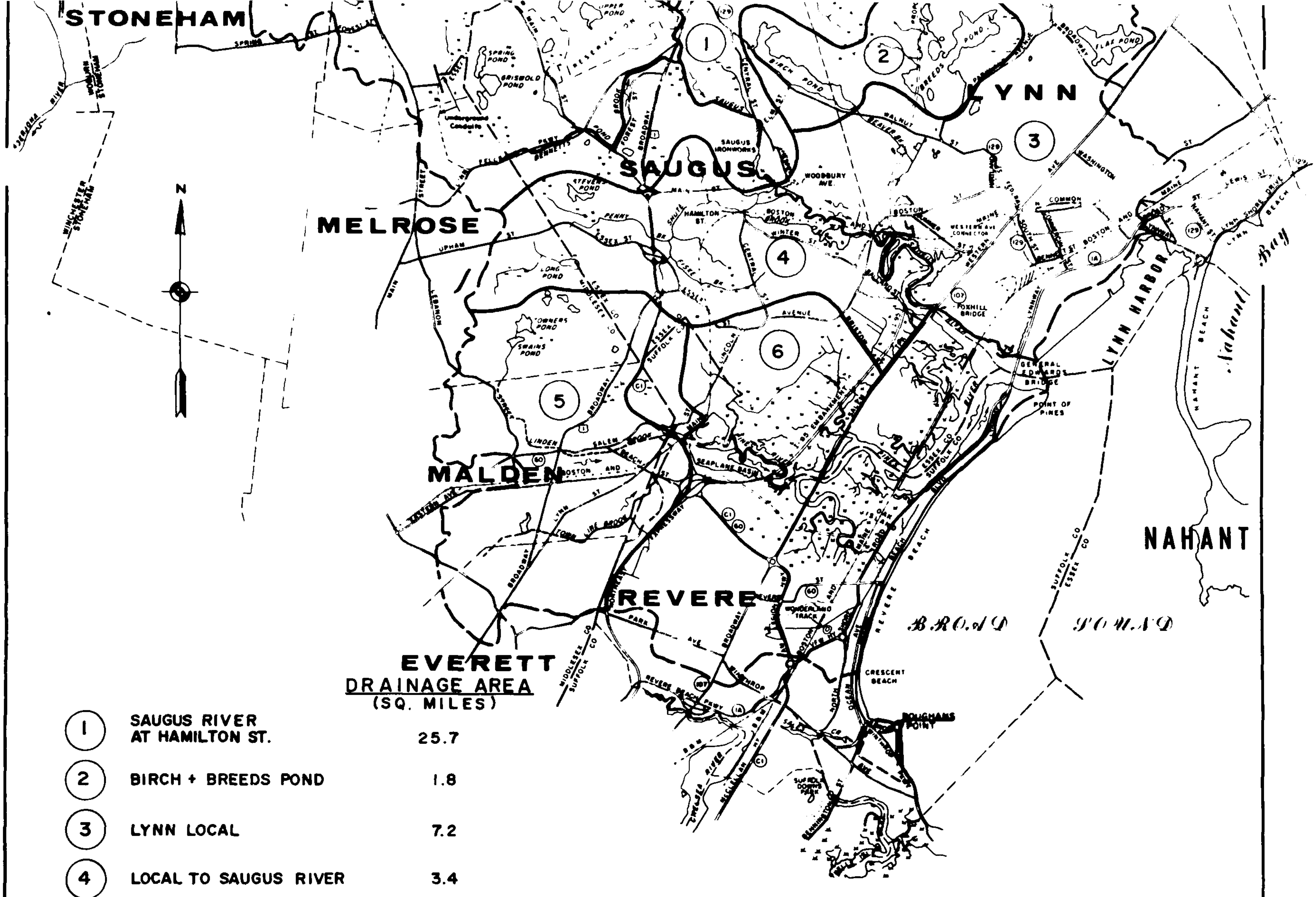
ID	1 Day Per Symbol Heading/Task	Resource
P1	SAGUS.PJ	
001	FIELD DATA	
002	EQUIP LEASE	
003	PREPARATION	
004	SURVEY	
005	CLEAN-UP	
006	SAMPLE ANALYSIS	
007	DATA REDUCTION	
008	FIELD DATA REP	
009	NUMERICAL MODEL	
010	DATA I D & ORG	
011	MESH DEVELOP	
012	VERIFICATION	
013	BASE & PLAN TEST	
014	SENSITIVITY TEST	
015	ANAL HYDRO	
016	SED TESTS	
017	SED ANALYSIS	
062	EROSION EVAL	
018	NUM MOD REP	
019	PHYSICAL MODEL	
050	DESIGN	
020	LAYOUT	
021	TEMPLATES	
022	PLOT TEMPLATES	
023	SHOP DRAWINGS	
024	ANAL FIELD DATA	
025	MODEL CONST	
026	E&CSD-SHOPS	
028	INSTRUMENTATION	
029	INSTALL WLD	
030	INSTALL METERS	
031	PHYS MOD VERIF	
032	LABOR	
034	BASE TESTS	
035	VELOCITIES	
036	ELEVATIONS	
037	PHOTOS	
038	MOSAICS	
039	PLAN TESTS	
040	VELOCITIES	
041	ELEVATIONS	
052	RIPRAP DESIGN	
042	PHOTOS	
043	MOSAICS	
044	ANALYZE RESULTS	
051	PHY MOD REP	
053	NAVIGATION MODEL	
054	UPDATE MODEL	
055	DESIGN & RECON	
056	DATA DEVELOPMENT	
057	TESTING	
058	ANALYSIS	
059	PRELIM RESULTS	
060	DRAFT REPORT	
061	NAV REPORT	



Non Critical » Milestone --- Float/Delay Finish Delay ___ Interrupt
Critical N Critical MS >>> Free Float Unassigned



CORPS OF ENGINEERS
U. S. ARMY
READING
LYNNFIELD
PEABODY
WAKEFIELD
STONEHAM
SAUGUS
LYNN
LAKE QUANNAPOWITT
VICINITY MAP
SCALE IN MILES
0 5 10 15 20
SAUGUS & PINES RIVERS WATERSHED
MASSACHUSETTS BAY
BOSTON
N.H.
MASS
LOCATION MAP
SCALE IN MILES
0 25 50 75 100 125 150
CANADA
ME
VT
NH
NY
MASS
CONN
RI
ATLANTIC OCEAN
MIDDLESEX COUNTY
ESSEX COUNTY
ROUTE 128
INTERSTATE
HIGHWAY I-93
BURBANK POND
SPRING POND
GRISWOLD POND
LOWER POND
UPPER POND
BIRCH POND
BREEDS POND
CEDAR POND
FLAX POND
LILY MERE



DRAINAGE AREA
(SQ. MILES)

1	SAUGUS RIVER AT HAMILTON ST.	25.7
2	BIRCH + BREEDS POND	1.8
3	LYNN LOCAL	7.2
4	LOCAL TO SAUGUS RIVER	3.4
5	LINDEN & TOWN LINE BRKS. (PINES RIVER INFLOW)	4.0
6	PINES RIVER LOCAL	4.9
	TOTAL:	47.0

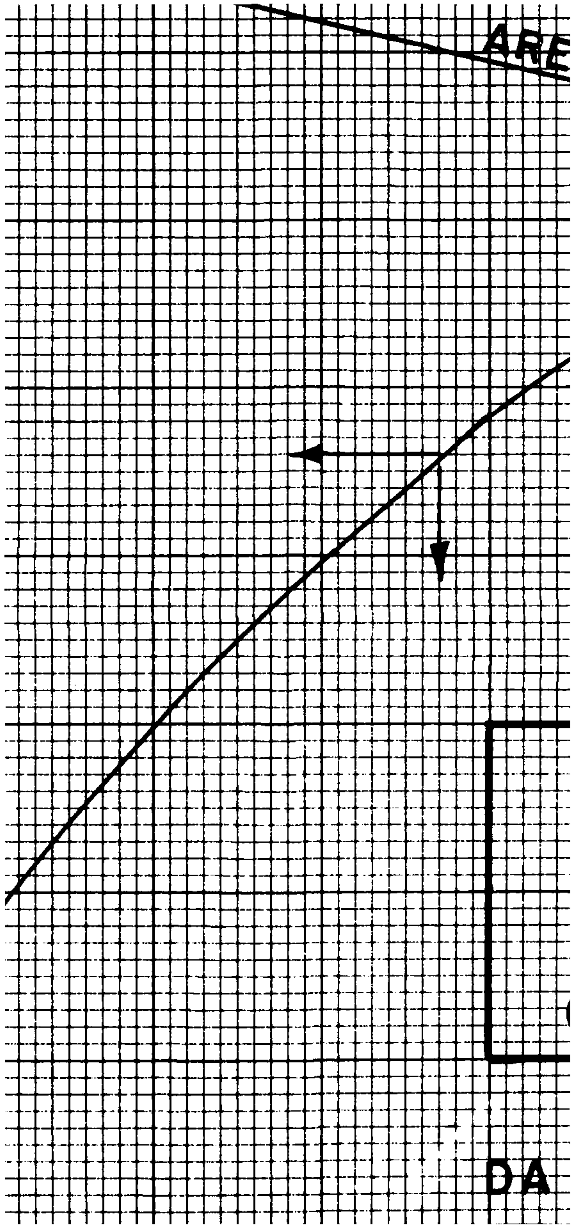
SAUGUS RIVER BASIN AND VICINITY

SAUGUS RIVER

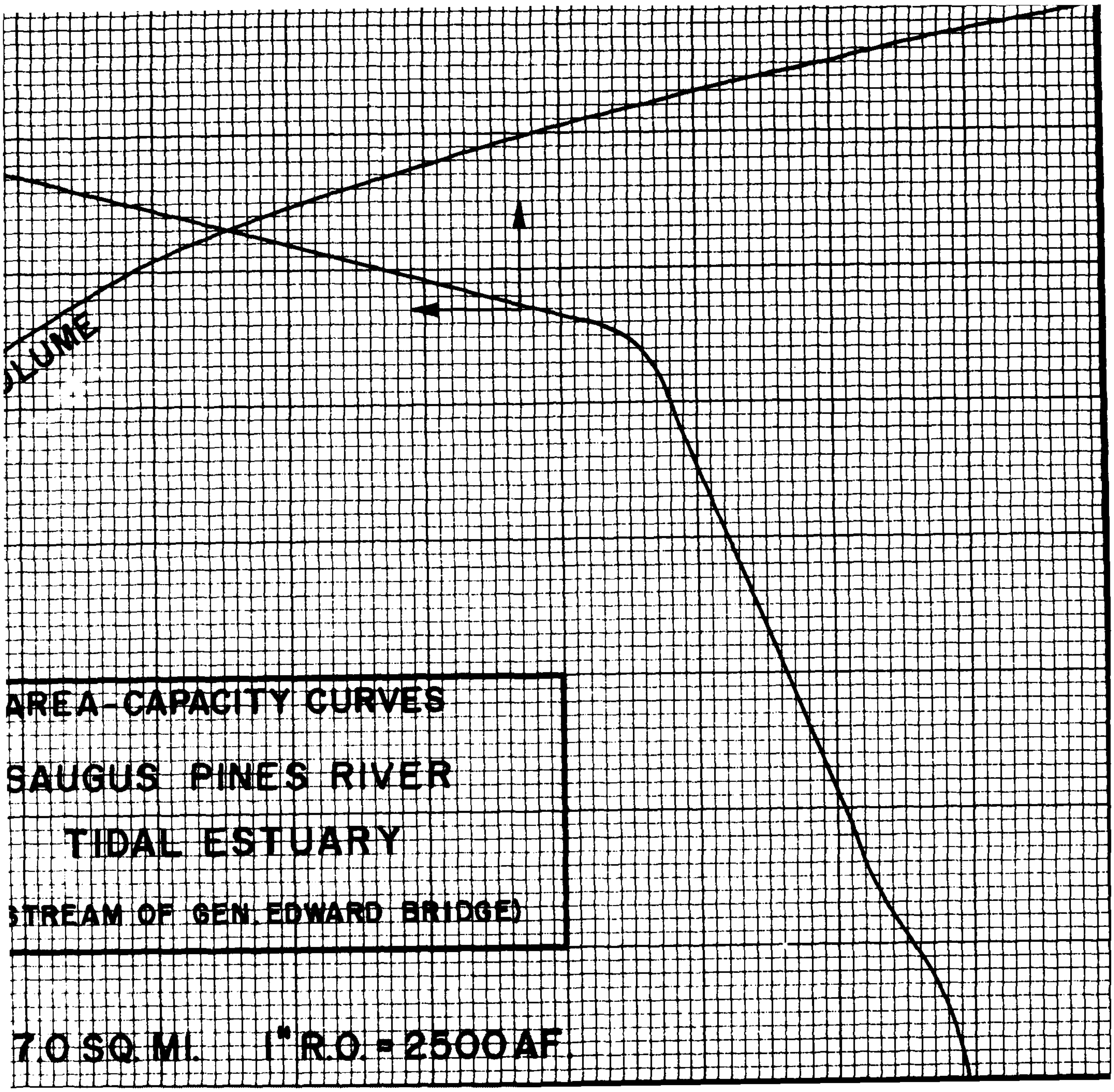
BASIN MAP

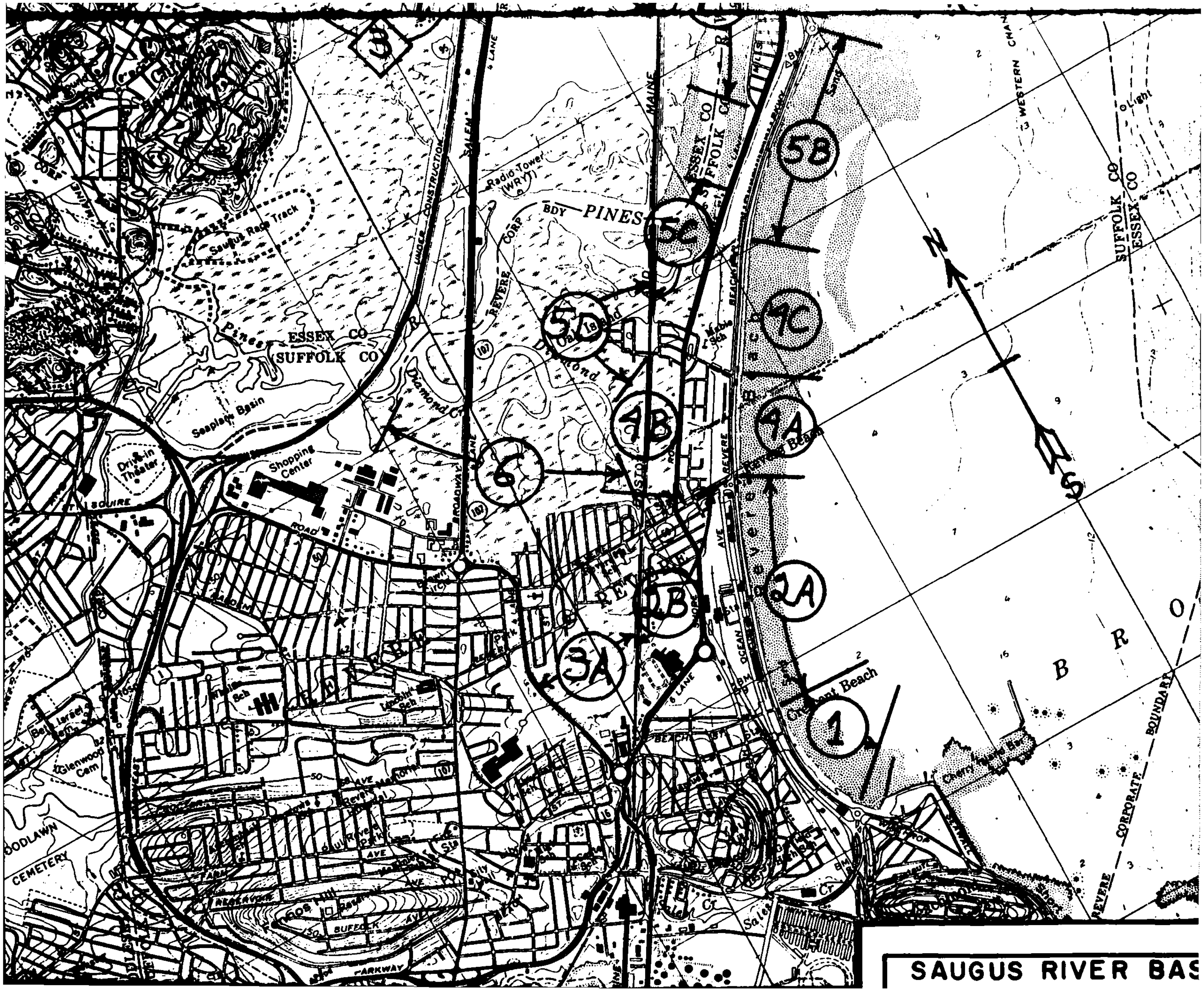
HYDRO. ENGR. SEC. APRIL 1988

PLATE I

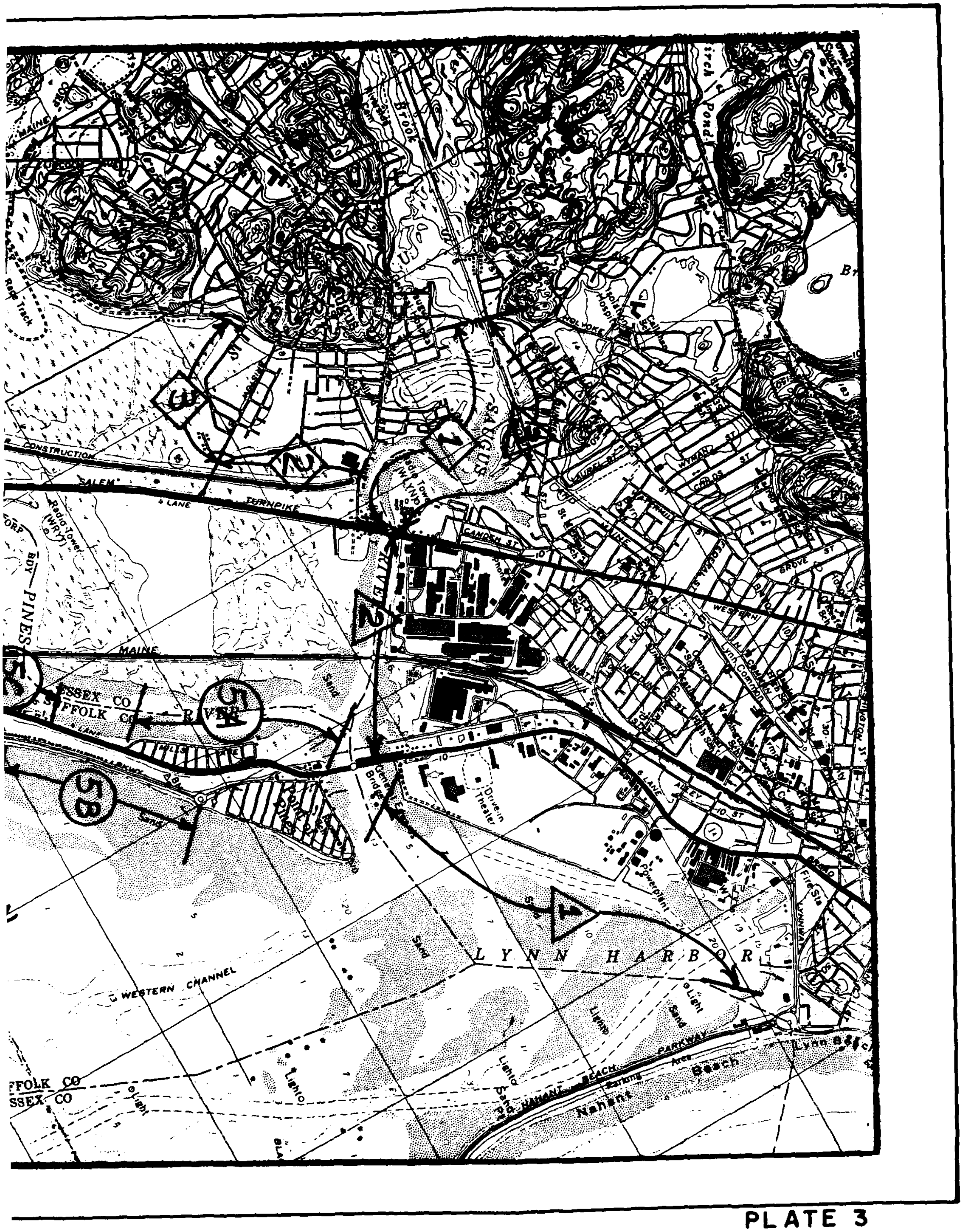


ARE
DA


LUME
AREA-CAPACITY CURVES
SAUGUS PINES RIVER
TIDAL ESTUARY
STREAM OF GEN. EDWARD BRIDGE)
7.0 SQ. MI.   1" R.O. = 2500 AF.

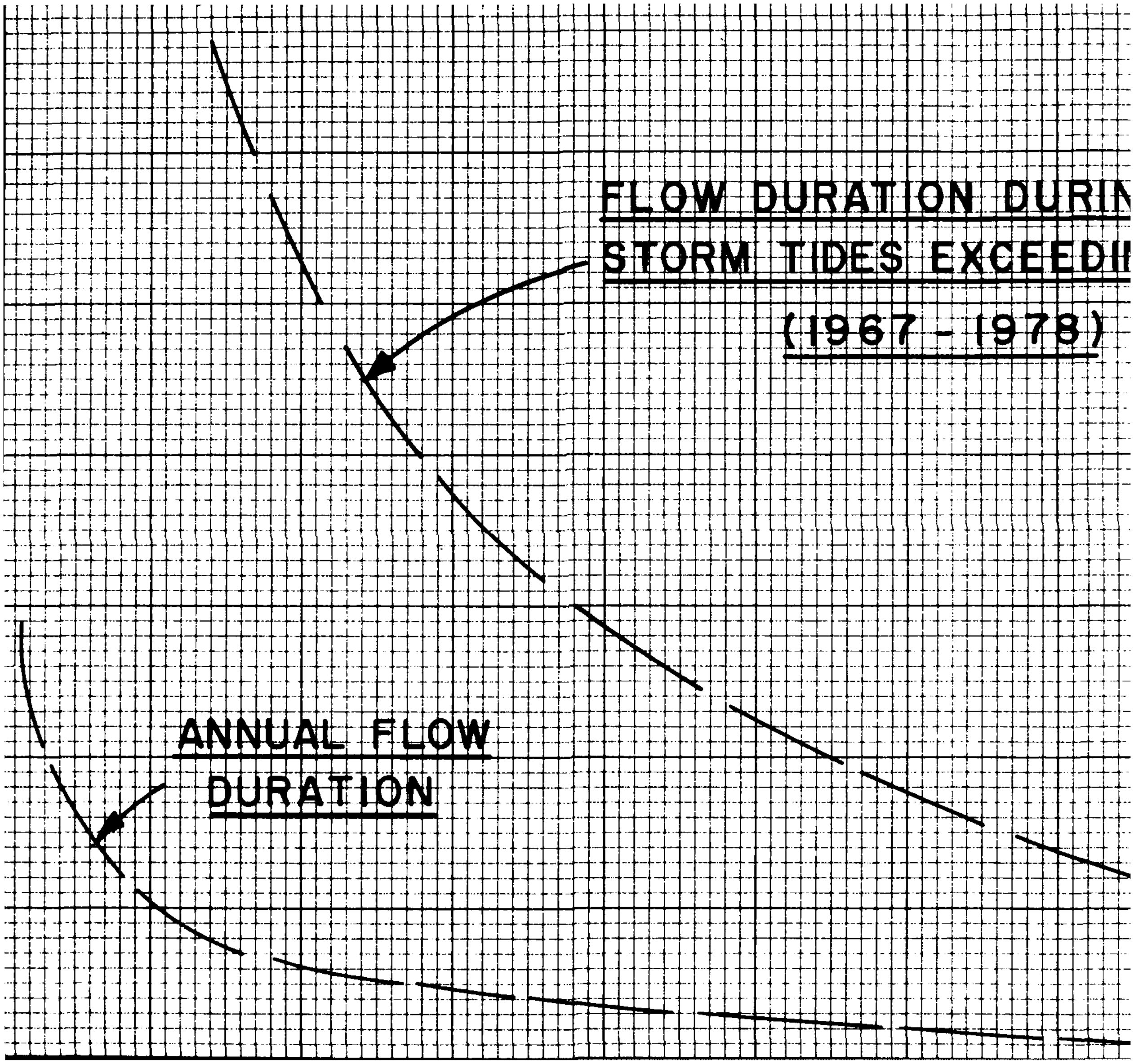


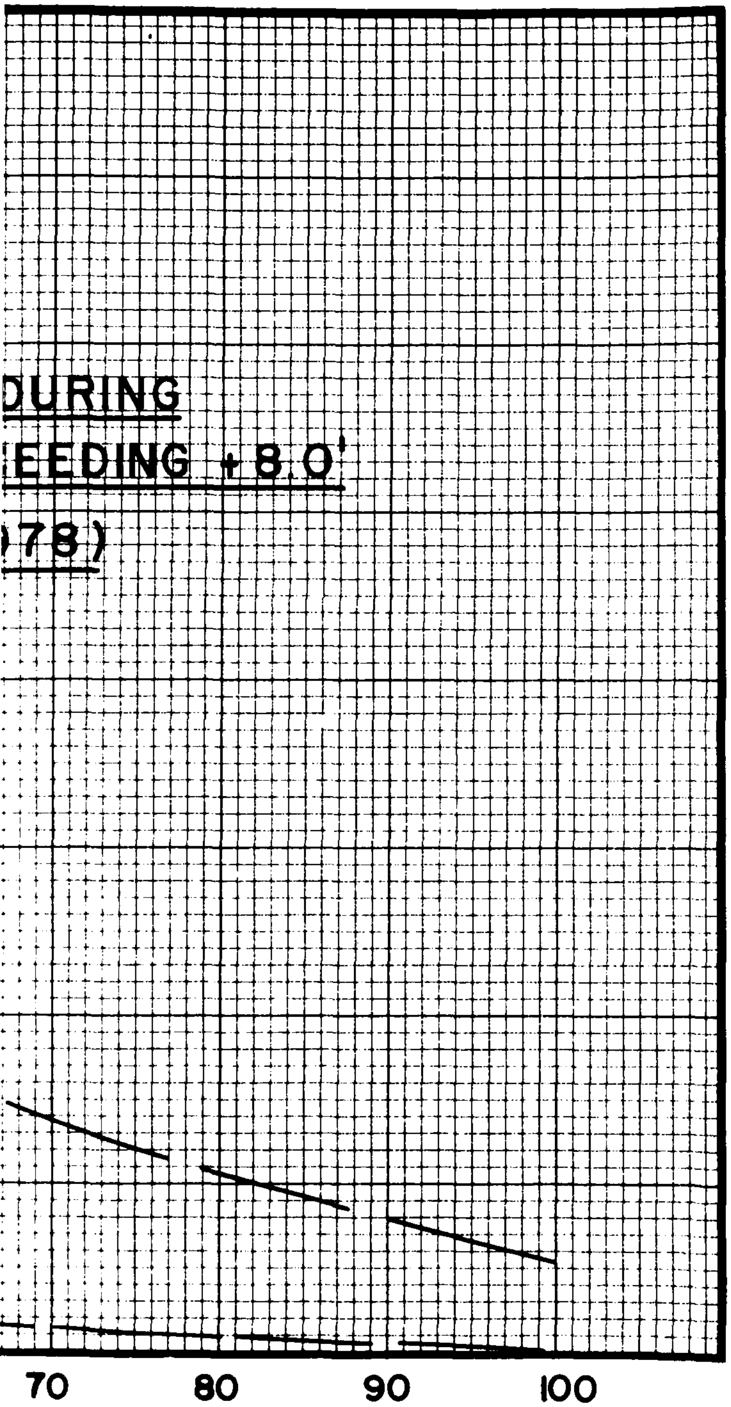
SAUGUS RIVER BAS



SALEM TURNPIKE
4 LANE
CONSTRUCTION
MAINE
ESSEX CO
SUFFOLK CO
RIVER
SAUGUS
PINES
LYNN HARBOR
WESTERN CHANNEL
NAHANT BEACH PARKWAY
Nahant Beach
Lynn Beach
Parking Area
Drive-in Theater
Lynn Common
WESTERN
WYMAN ST
CAMDEN ST
GROVE ST
HOLYOKE
Brook
Pond
Sand
Light
Race Track
Radio Tower
Power plant
5B

PLATE 3


FLOW DURATION DURIN
STORM TIDES EXCEEDI
(1967 - 1978)
ANNUAL FLOW
DURATION



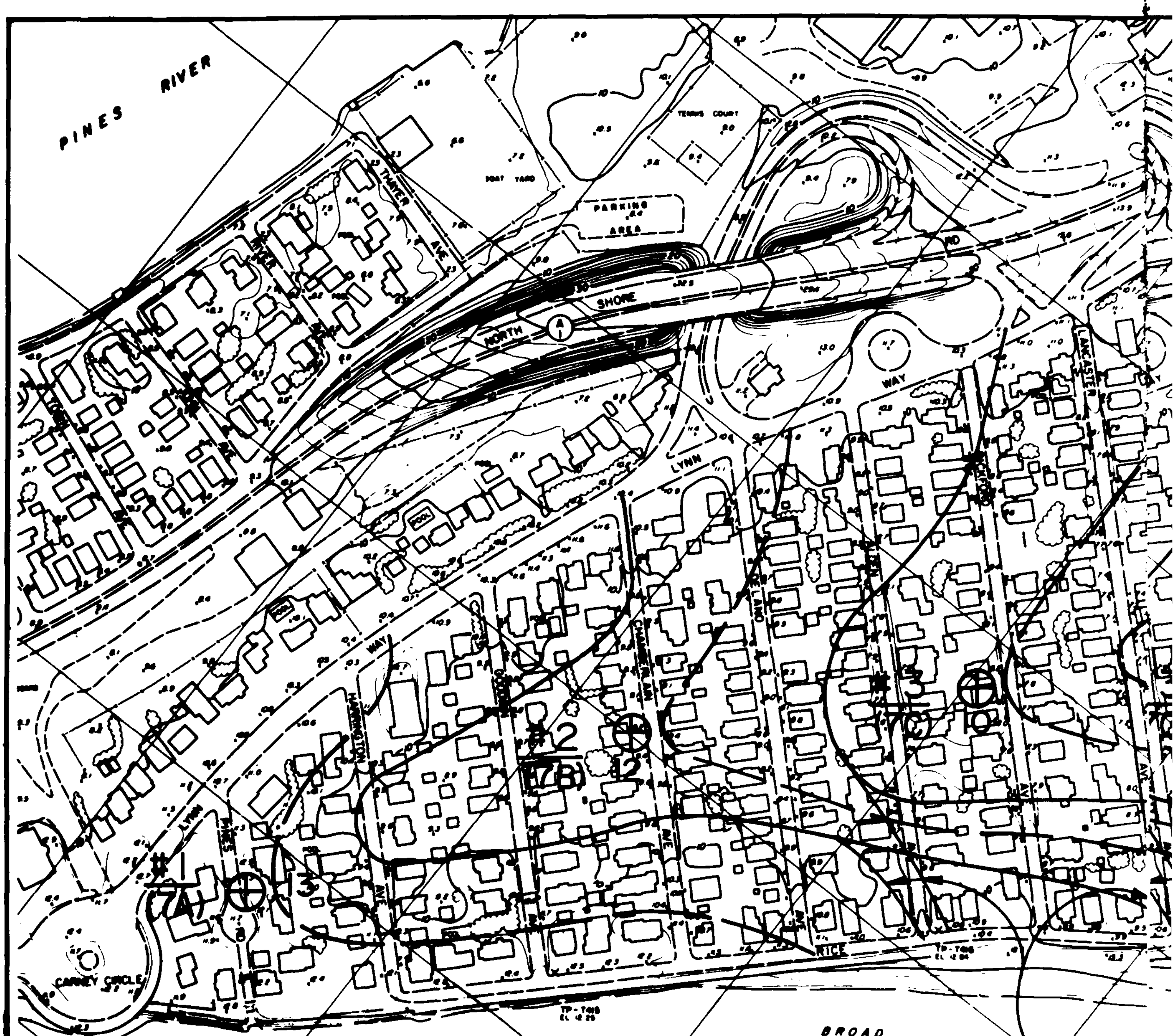
CEEDED

OLD SWAMP RIVER NR.
SO. WEYMOUTH, MASS.
D.A. = 4.5 SQ. MI.

NORMAL & HIGH TIDE FLOW DURATIONS

HYDRO. ENGR. APRIL 1988

PLATE 4



SOU

NEW LIMI

OVERTO

LEGEND

#2 ZONE NUMBER

INDEX LINE EL. ⊕10

LIMIT OF ZONE

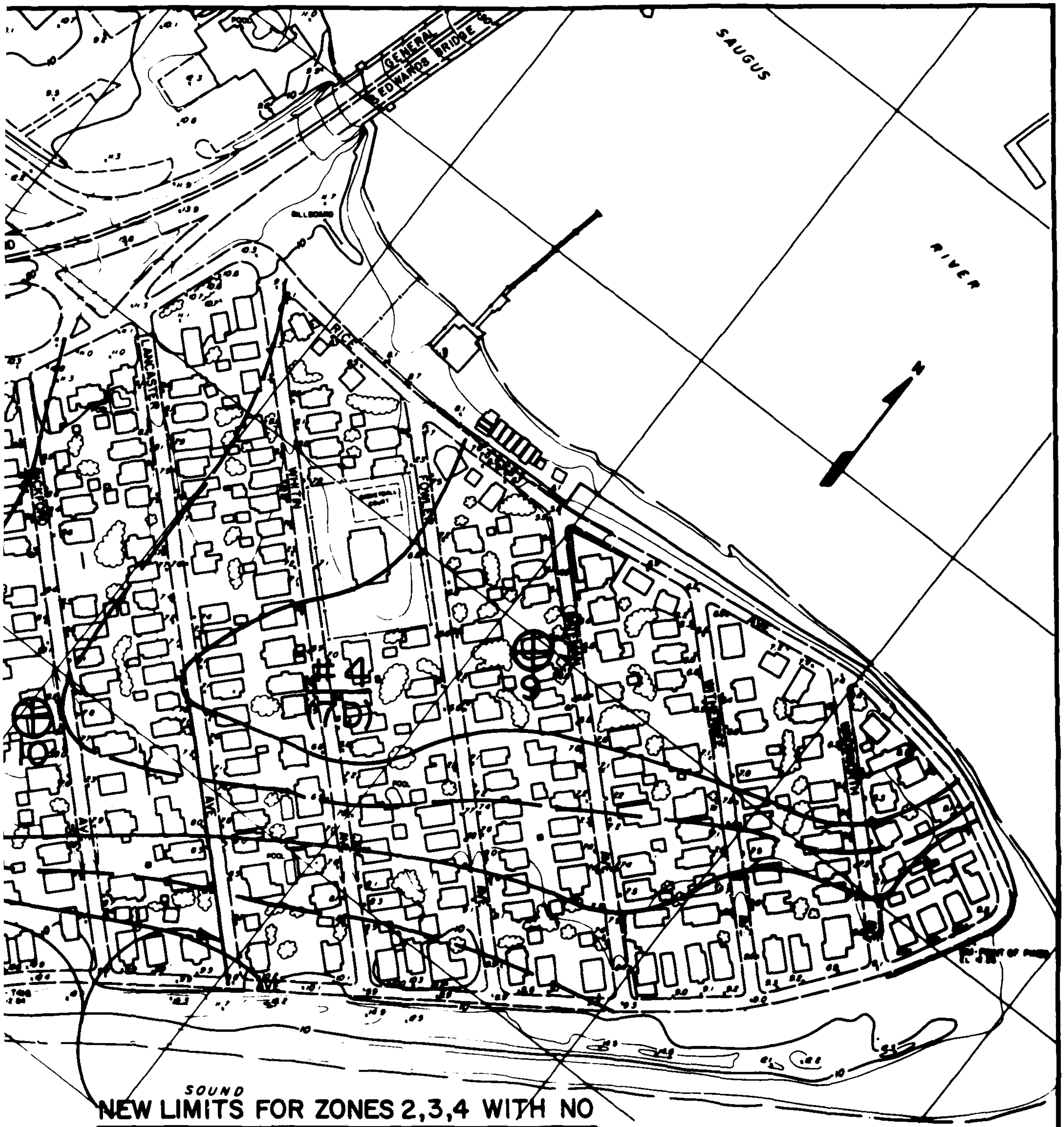
SCALE

100 0 100 200 300 400

500 FOO
CHUSETT

NATIONA
OF 1929

DATE OF
CONTOUR



NEW LIMITS FOR ZONES 2,3,4 WITH NO OVERTOPPING ALONG REACH E.

500 FOOT GRID BASED UPON MASSACHUSETTS RECTANGULAR GRID SYSTEM

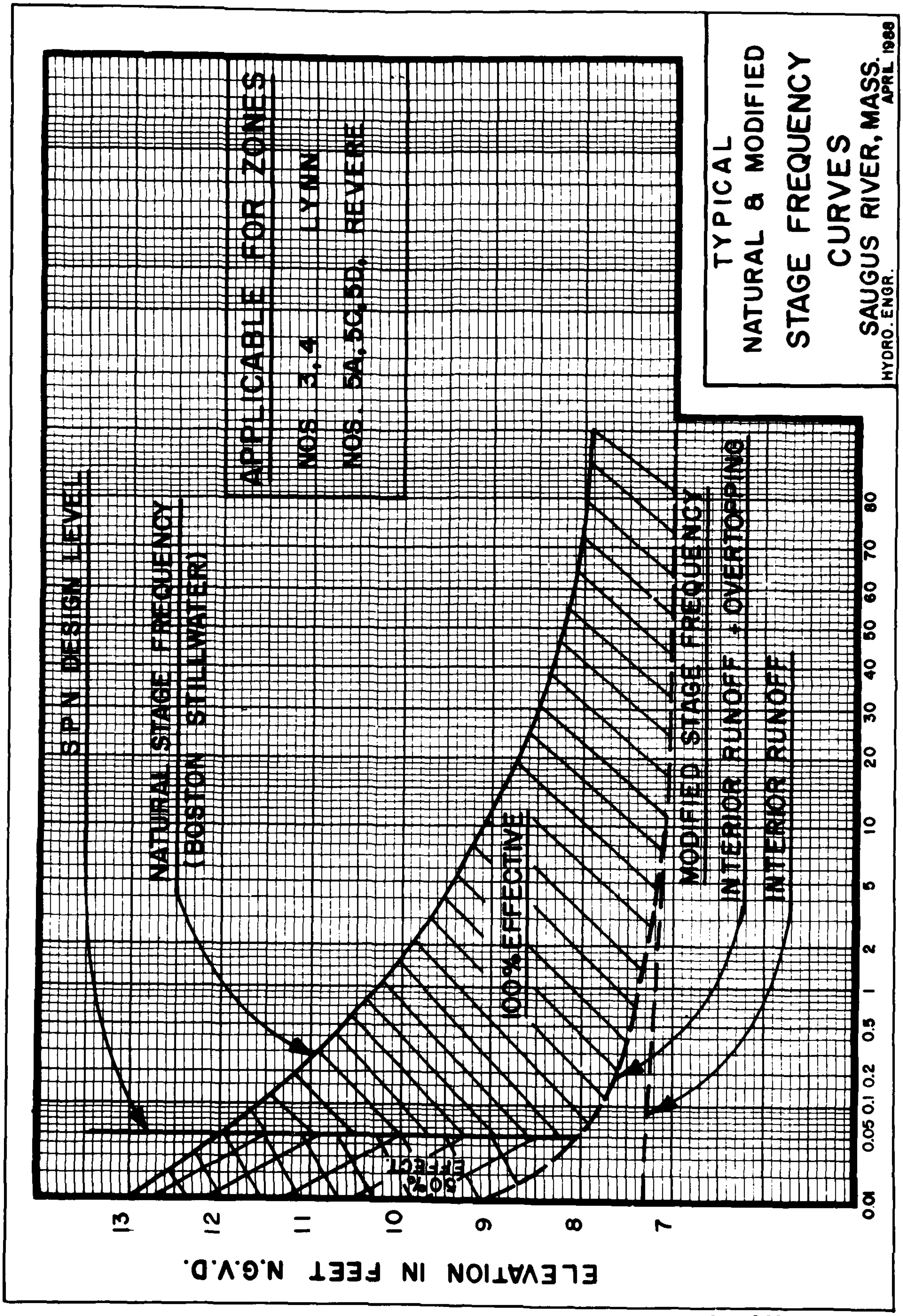
NATIONAL GEODETIC VERTICAL DATUM OF 1929

DATE OF PHOTOGRAPHY - 2-7-81
CONTOUR INTERVAL - 2 FT.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

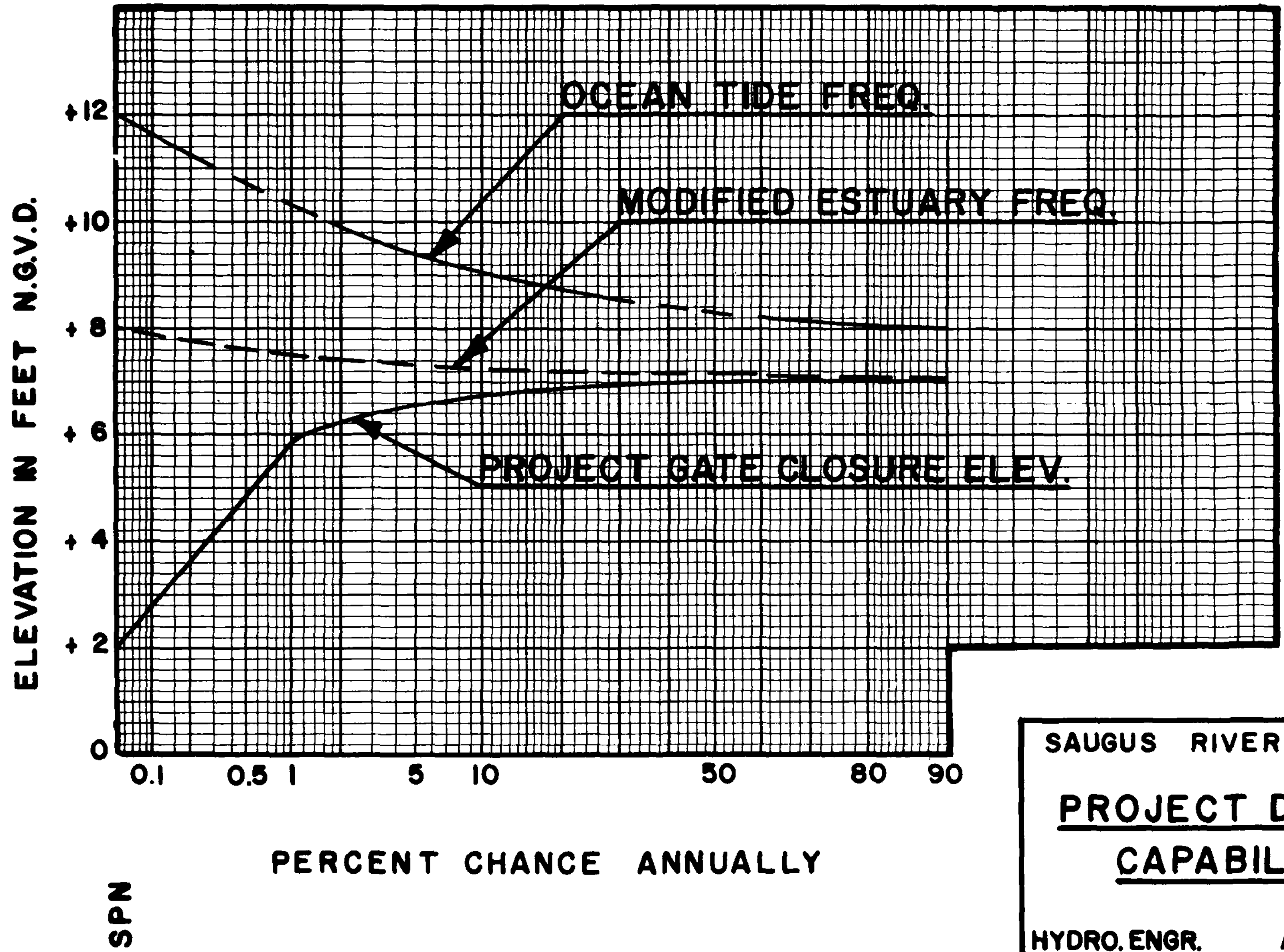
COASTAL FLOOD PROTECTION
POINT OF PINES, REVERE
MASSACHUSETTS
INDEX ZONE LIMITS

PLATE 5



TYPICAL
NATURAL & MODIFIED
STAGE FREQUENCY
CURVES
SAUGUS RIVER, MASS.
HYDRO. ENGR. APRIL 1968

PLATE 6



SAUGUS RIVER FLOODGATE

PROJECT DESIGN CAPABILITY

HYDRO. ENGR. APRIL 1988

PLATE 7

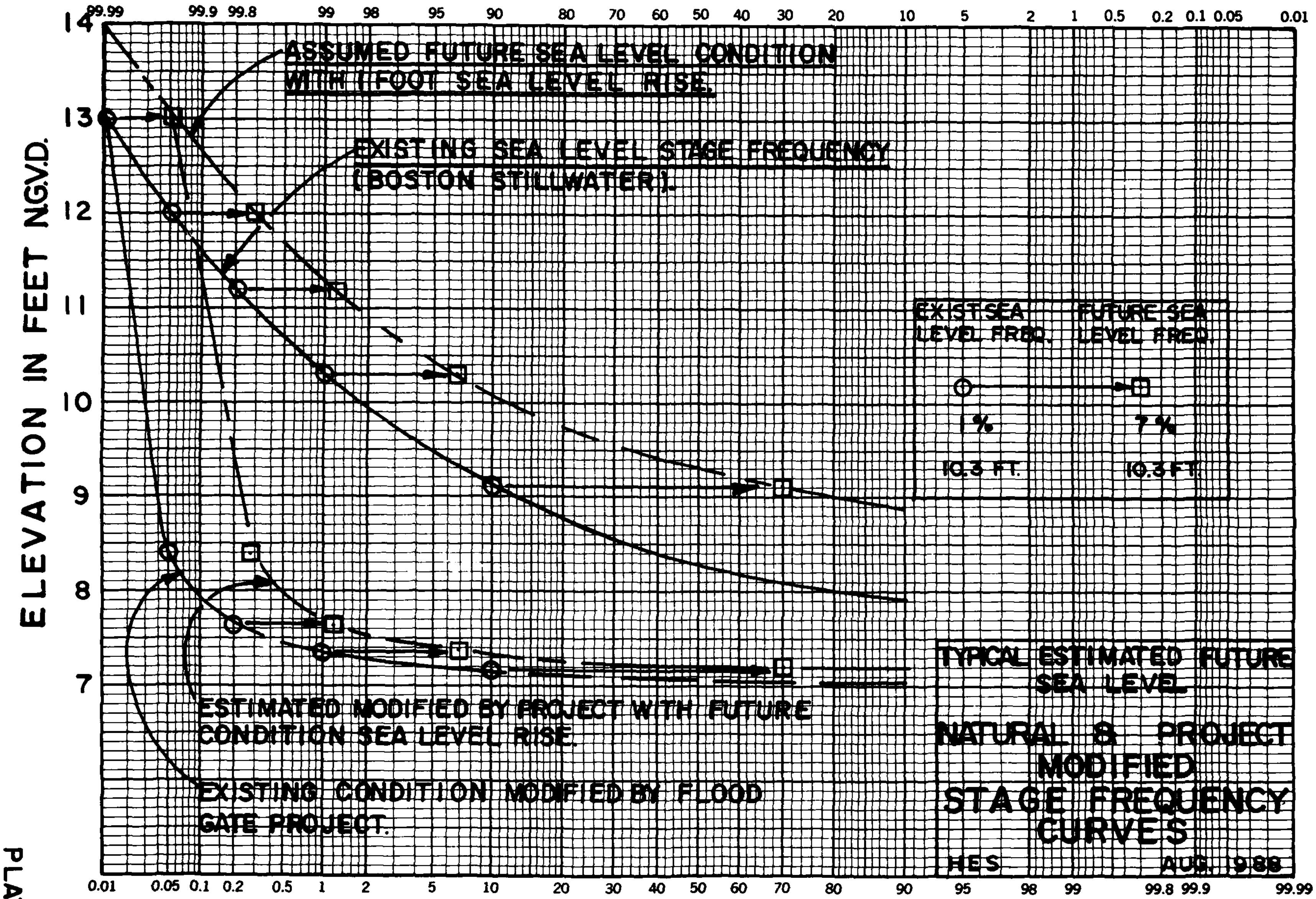
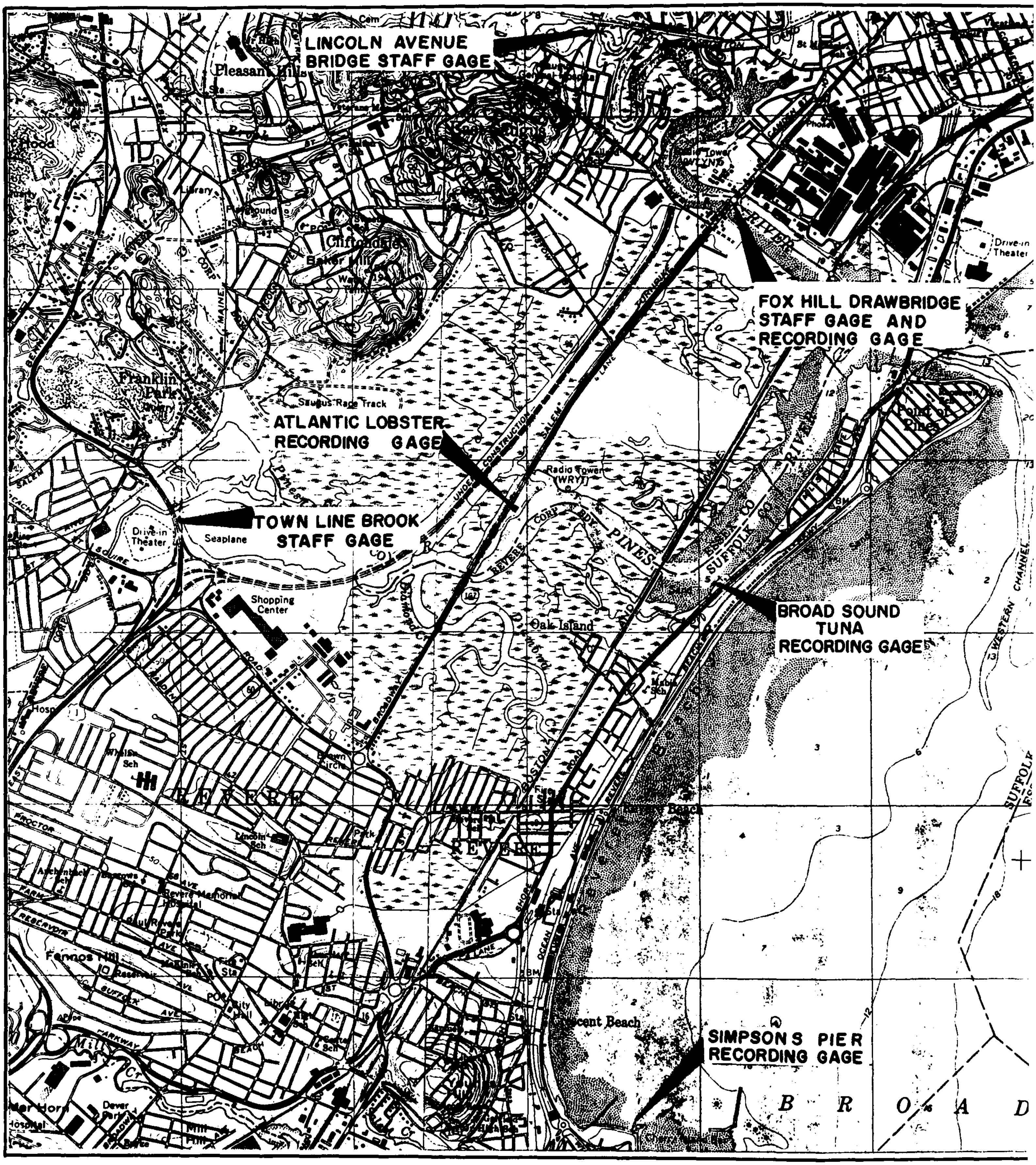
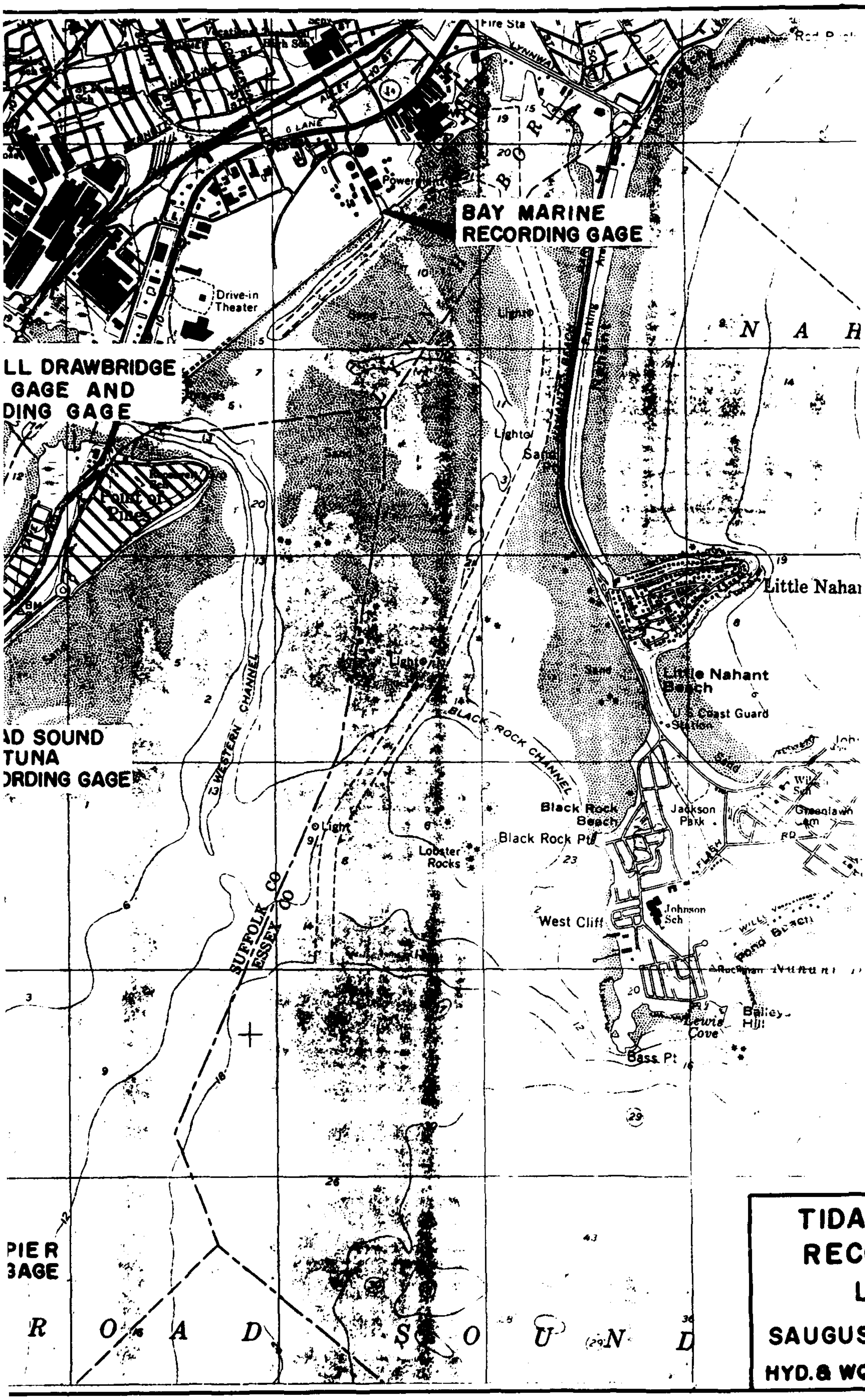

ASSUMED FUTURE SEA LEVEL CONDITION WITH 1 FOOT SEA LEVEL RISE.
EXISTING SEA LEVEL STAGE FREQUENCY (BOSTON STILLWATER).
EXIST SEA LEVEL FREQ.
FUTURE SEA LEVEL FREQ.
1%
7%
10.3 FT.
10.3 FT.
ESTIMATED MODIFIED BY PROJECT WITH FUTURE CONDITION SEA LEVEL RISE.
EXISTING CONDITION MODIFIED BY FLOOD GATE PROJECT.
TYPICAL ESTIMATED FUTURE SEA LEVEL
NATURAL & PROJECT MODIFIED STAGE FREQUENCY CURVES
HES
AUG. 1988
ELEVATION IN FEET N.G.V.D.
7
8
9
10
11
12
13
14
99.99
99.9
99.8
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
0.05
0.01

PLATE 8


LINCOLN AVENUE BRIDGE STAFF GAGE
FOX HILL DRAWBRIDGE STAFF GAGE AND RECORDING GAGE
ATLANTIC LOBSTER RECORDING GAGE
TOWN LINE BROOK STAFF GAGE
BROAD SOUND TUNA RECORDING GAGE
SIMPSONS PIER RECORDING GAGE
Pleasant Hill
Cliftondale
Franklin Park
Saugus Race Track
Drive-in Theater
Seaplane
Shopping Center
Oak Island
Radio Tower (WRYT)
REVERE
Revere Memorial Hospital
Fennos Hill
Crescent Beach
Point of Pines
B R O A D

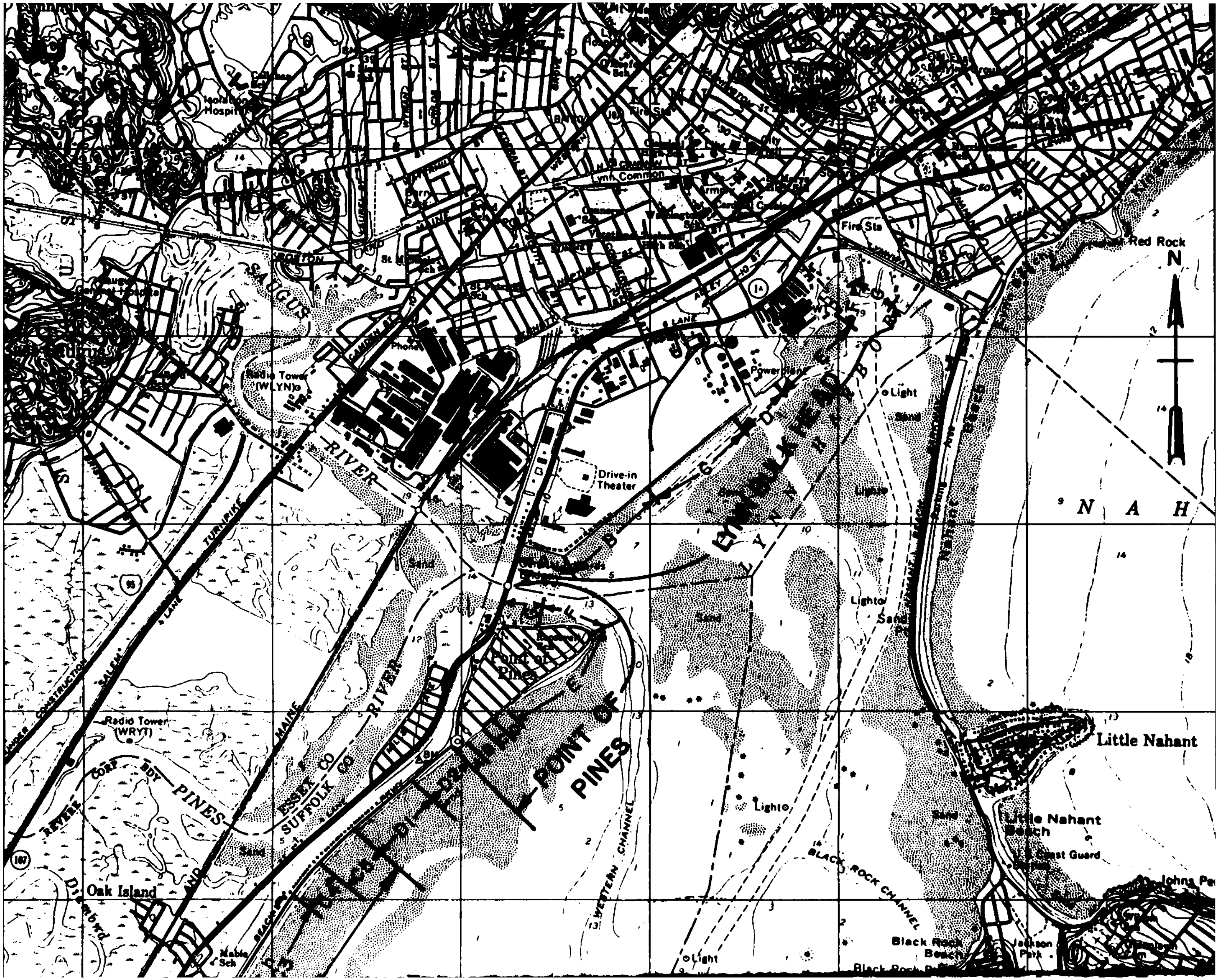


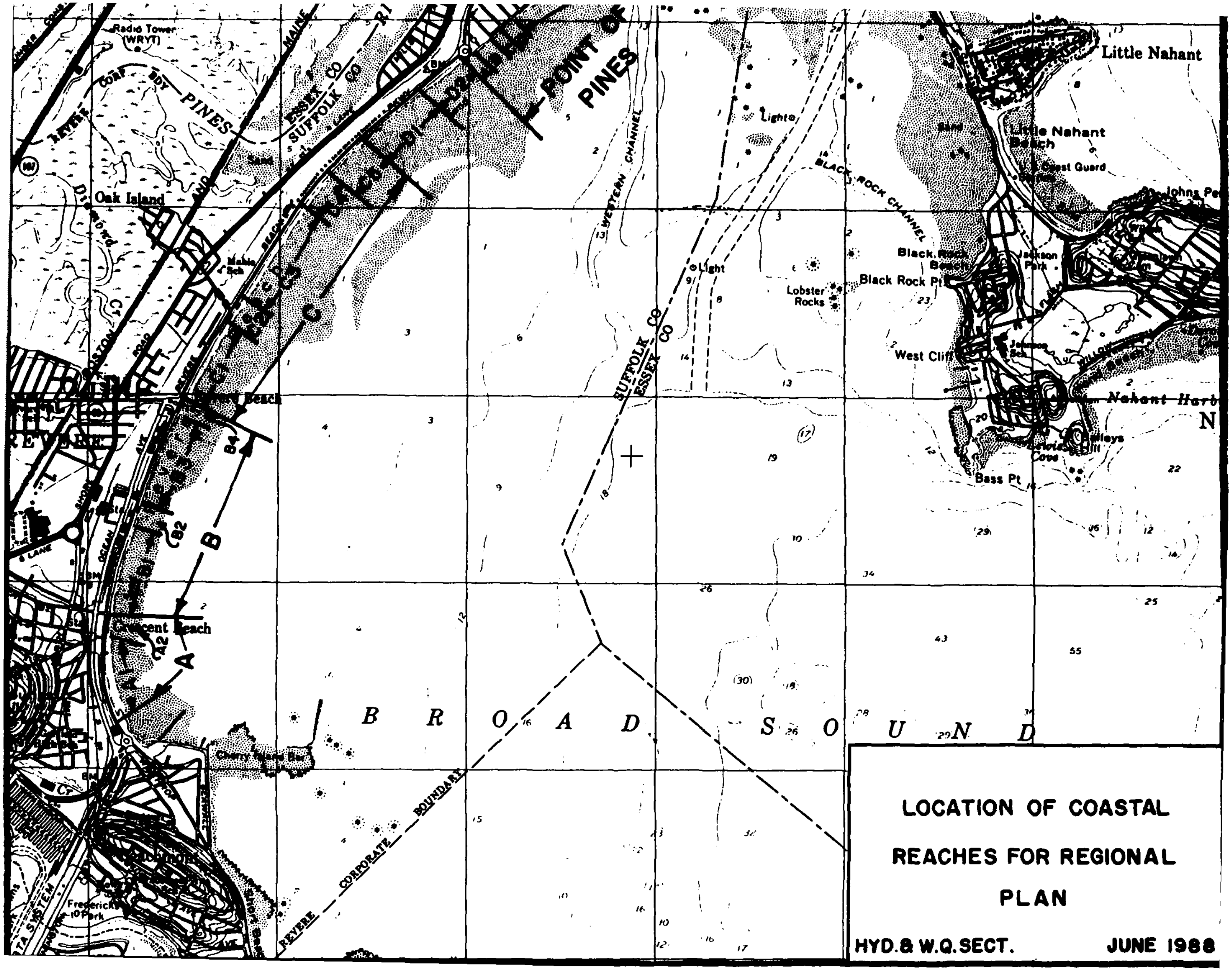
TIDAL STAFF AND RECORDING GAGE LOCATIONS

SAUGUS RIVER ESTUARY

HYD.& WQ.SECT. JUNE 1968

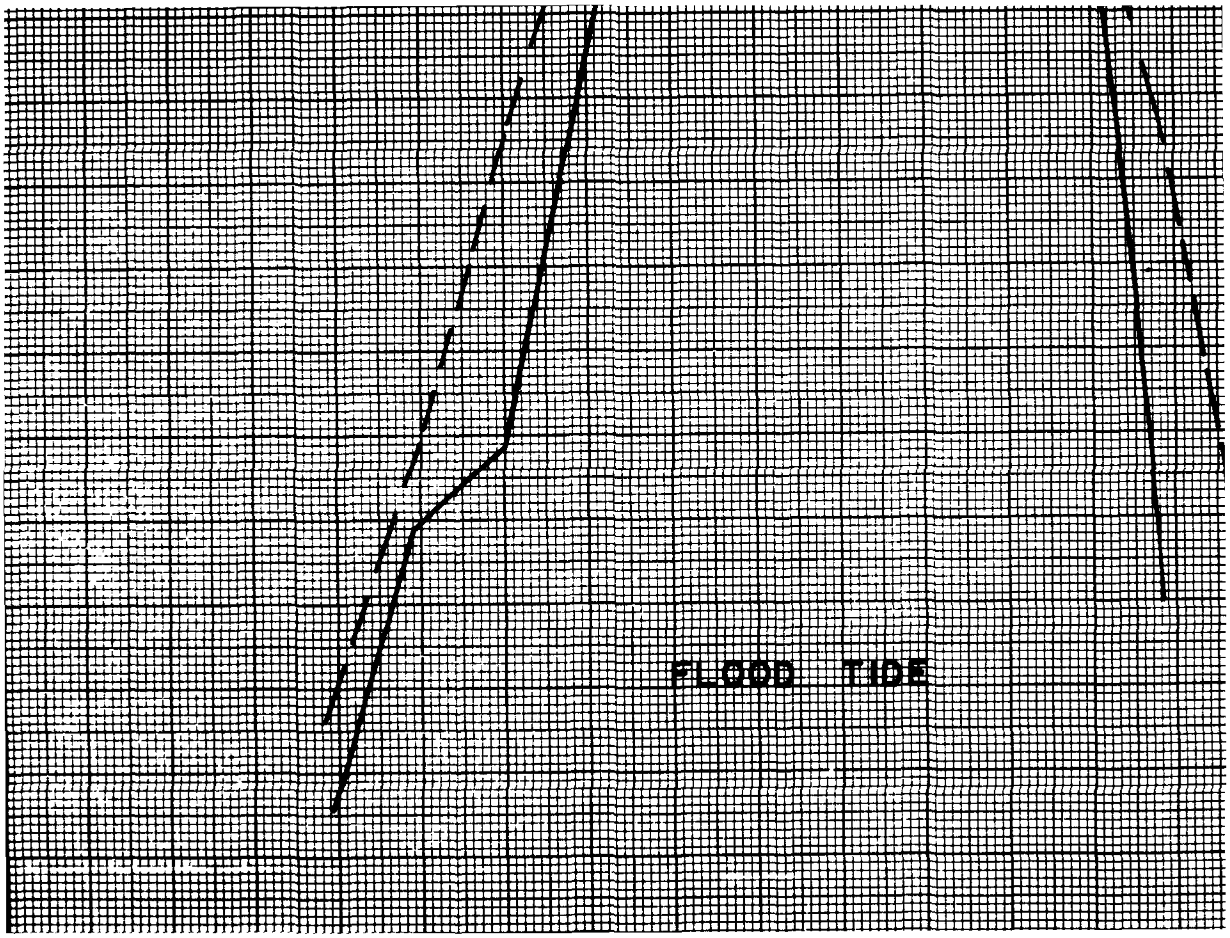
PLATE 9

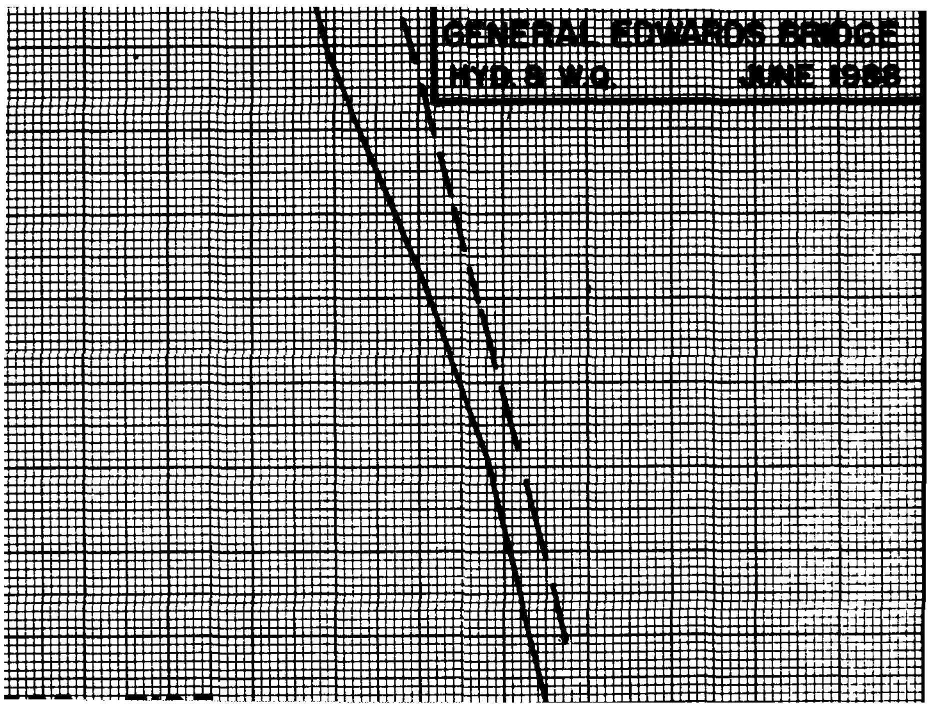

Red Rock
N
NAH
Little Nahant
Little Nahant Beach
Black Rock Beach
Jackson Park
BLACK ROCK CHANNEL
WESTERN CHANNEL
POINT OF PINES
Point of Pines
Drive-in Theater
Lynn Common
Fire Sta
Radio Tower (WLYN)
Radio Tower (WRYT)
SAUGUS
RIVER
ESSEX CO
SUFFOLK CO
PINES
Oak Island
Light
Sand

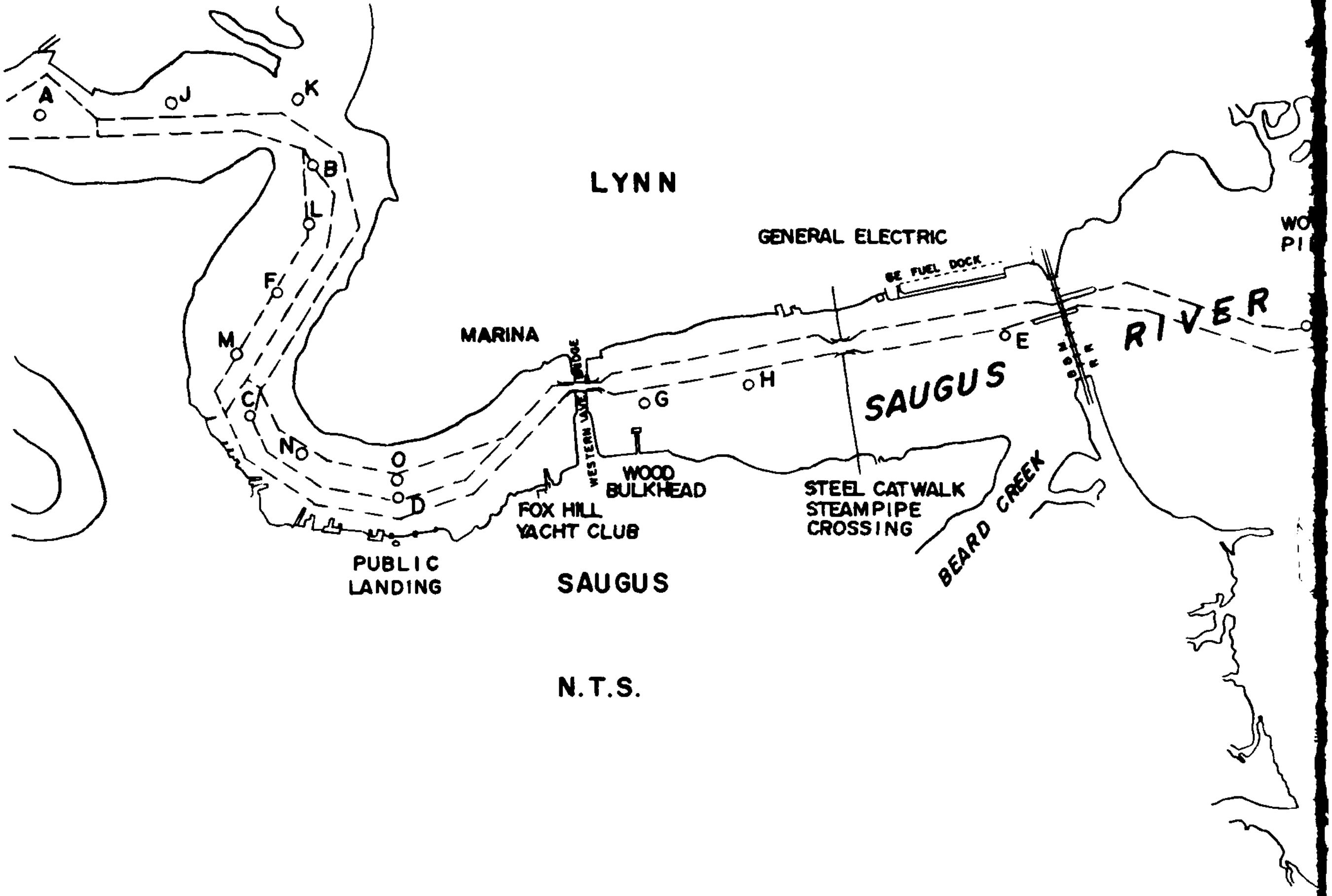


LOCATION OF COASTAL REACHES FOR REGIONAL PLAN

HYD.& W.Q.SECT. JUNE 1988


FLOOD TIDE


GENERAL EDWARDS BRIDGE
HYD. & W.Q.
JUNE 1988



LEGEND

AO = LOCATION OF SEDIMENT SAMPLE.

NOTE:

SAMPLES Q, R, AND S LOCATED IN THE WESTERN CHANNEL IN LYNN HARBOR.

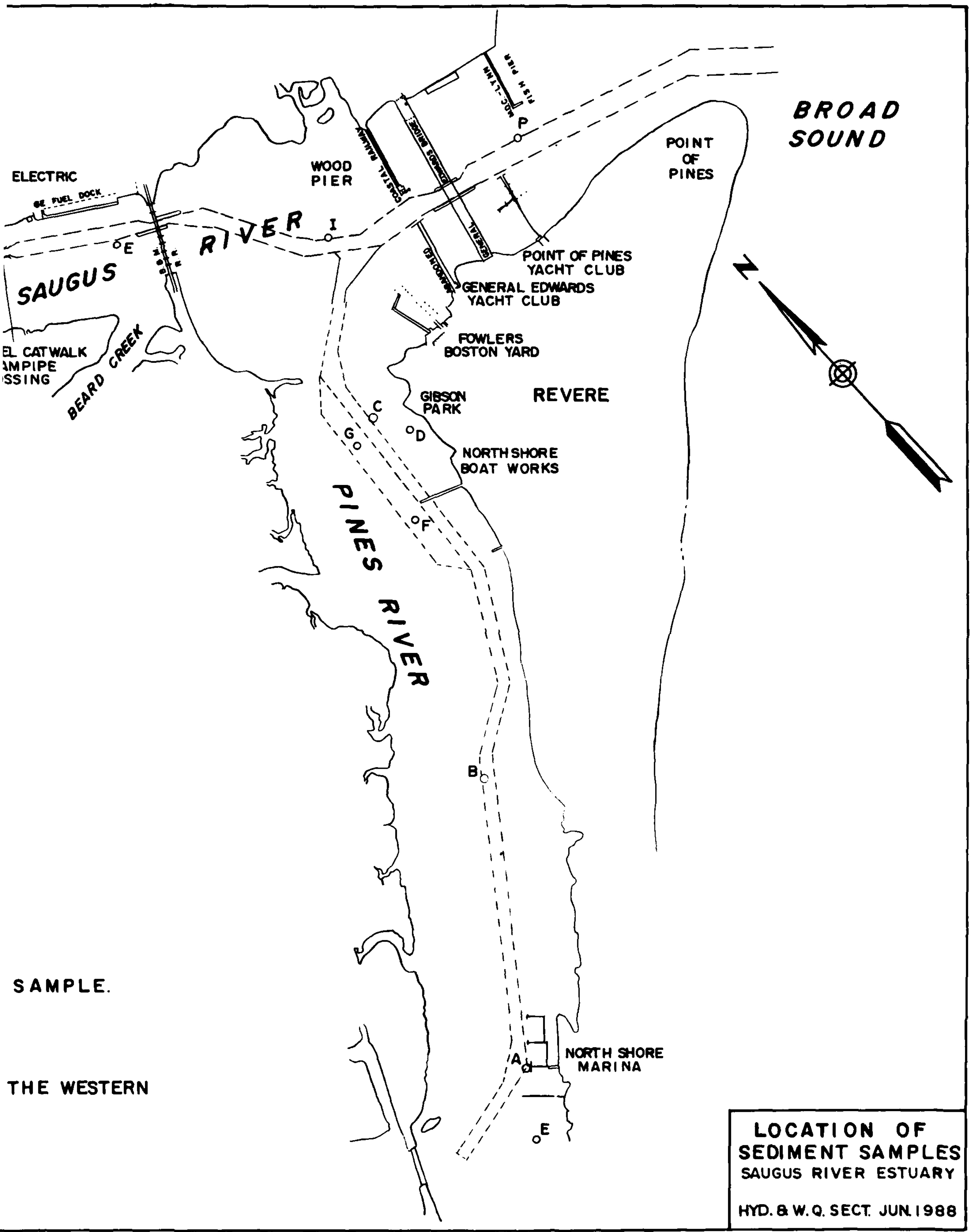
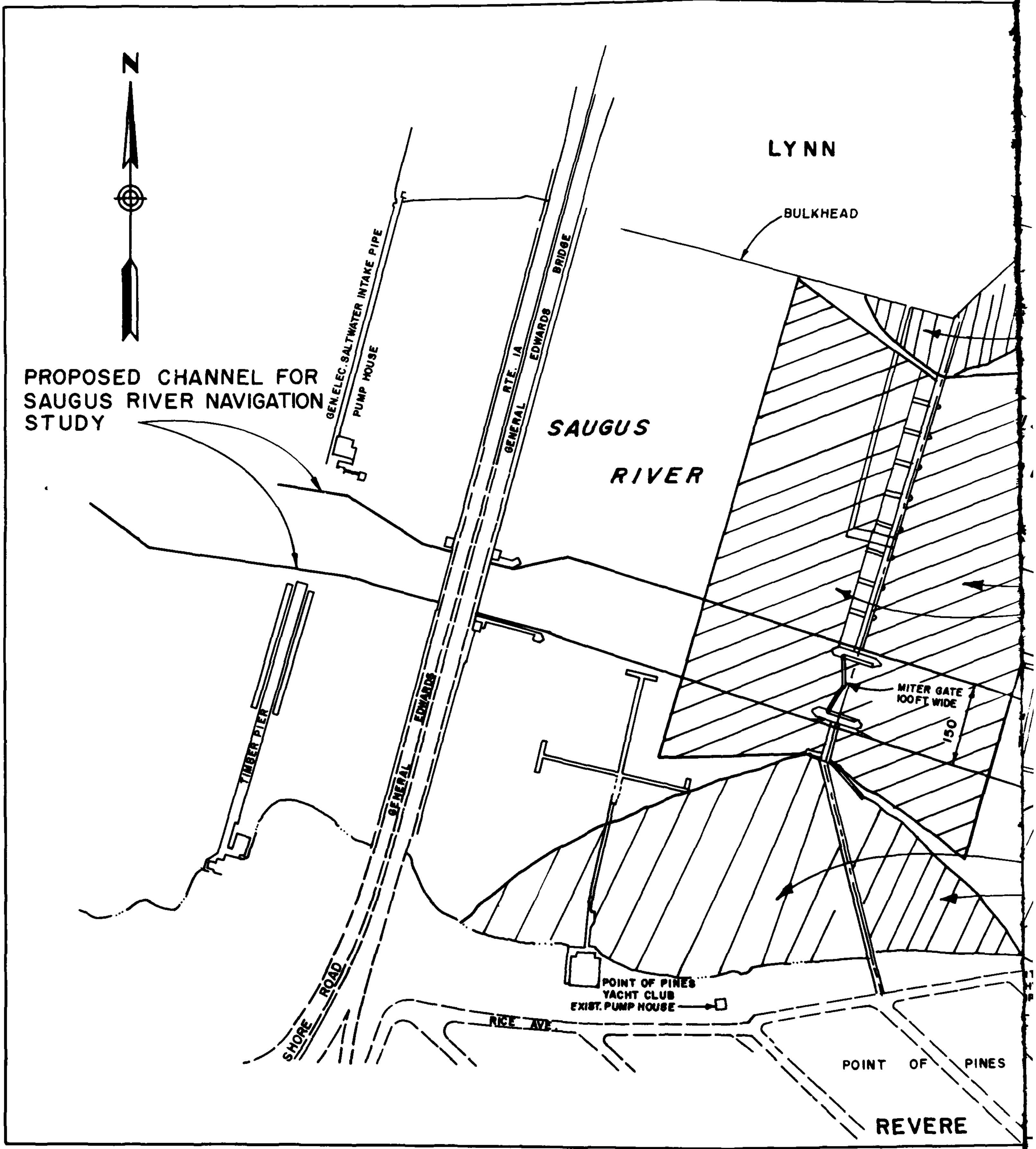
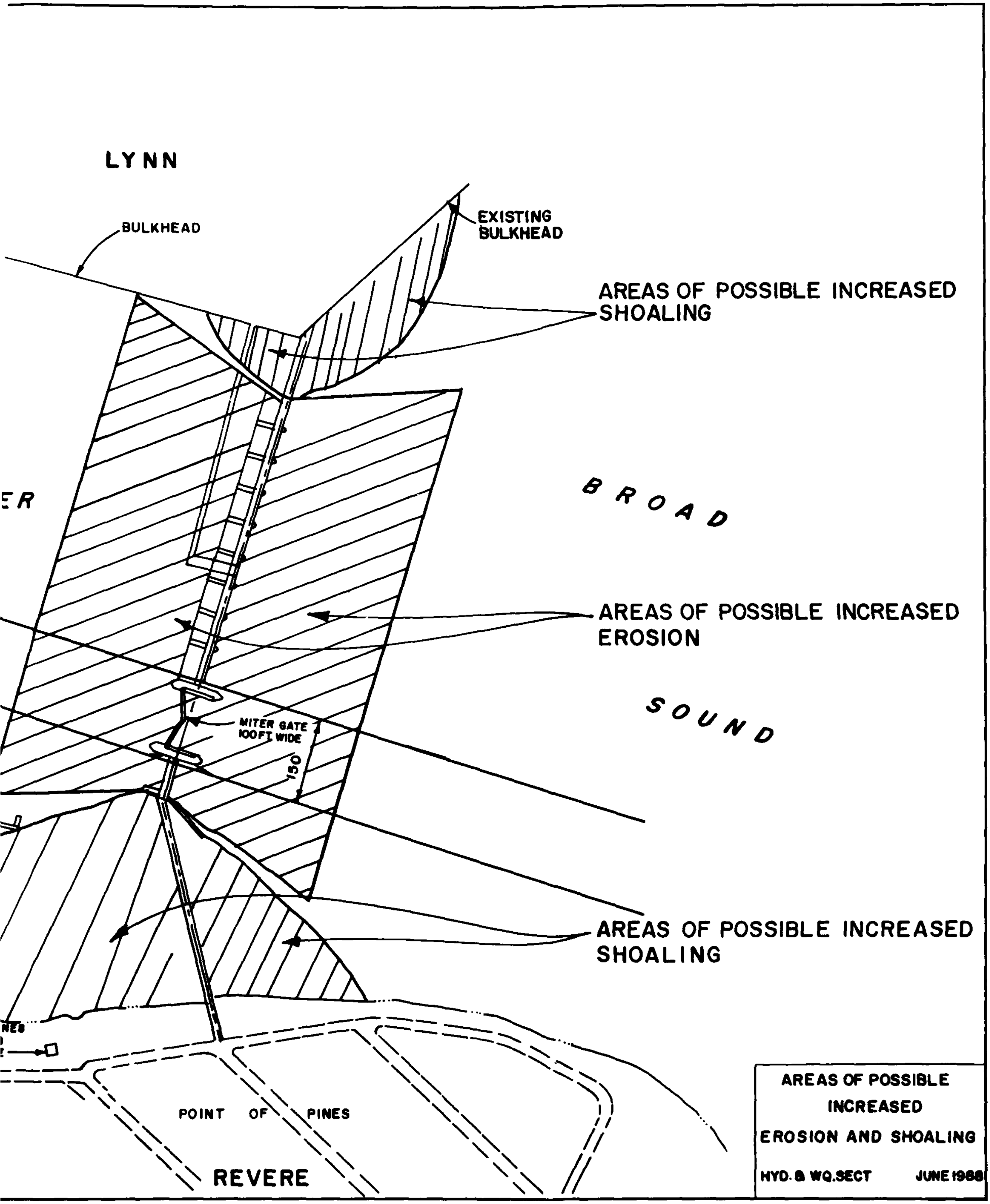

BROAD SOUND
POINT OF PINES
WOOD PIER
ELECTRIC
FUEL DOCK
P
I
E
SAUGUS RIVER
EL CATWALK
AMPIPE
SSING
BEARD CREEK
POINT OF PINES YACHT CLUB
GENERAL EDWARDS YACHT CLUB
FOWLERS BOSTON YARD
GIBSON PARK
REVERE
C
G
D
NORTH SHORE BOAT WORKS
F
PINES RIVER
B
SAMPLE.
THE WESTERN
A
NORTH SHORE MARINA
E
LOCATION OF SEDIMENT SAMPLES
SAUGUS RIVER ESTUARY
HYD. & W. Q. SECT. JUN. 1988

PLATE 12



N
LYNN
BULKHEAD
PROPOSED CHANNEL FOR
SAUGUS RIVER NAVIGATION
STUDY
GEN. ELEC. SALTWATER INTAKE PIPE
PUMP HOUSE
RTE. 1A
GENERAL EDWARDS BRIDGE
SAUGUS
RIVER
TIMBER PIER
GENERAL EDWARDS
MITER GATE
100 FT. WIDE
150
SHORE ROAD
POINT OF PINES
YACHT CLUB
EXIST. PUMP HOUSE
RICE AVE.
POINT OF PINES
REVERE



AREAS OF POSSIBLE INCREASED EROSION AND SHOALING

HYD. & WQ. SECT JUNE 1988

PLATE 13

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN, MALDEN, REVERE, AND SAUGUS
MASSACHUSETTS

WATER QUALITY

APPENDIX C

Hydraulics and Water Quality Section
Water Control Branch
Engineering Division

Department of the Army
New England Division, Corps of Engineers
424 Trapelo Road
Waltham, Massachusetts 02254-9149

June 1989

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN, MALDEN, REVERE AND SAUGUS
MASSACHUSETTS

TABLE OF CONTENTS

Paragraph	Subject	Page
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SUMMARY

1. As part of the flood damage reduction investigation for the Saugus River and tributaries, an analysis of water quality conditions in the Saugus and Pines Rivers estuary was conducted and the results are presented in this report.

2. The estuary's flow pattern, which has a major impact on water quality, is dominated by tidal movement since the freshwater component is such a small portion of the overall water volume within the estuary.

3. Existing water quality conditions were evaluated. Although the Saugus and Pines Rivers estuary generally meets Massachusetts water quality standards during non-rainfall periods, there are a significant number of exceedances of coliform bacteria criteria during runoff events. Also, dissolved oxygen criteria are not met in the upper portions of the estuary on occasion.

4. Of the three flood reduction options (described in detail in the Main Report) evaluated, the local protection project option and the nonstructural option have only minimal impacts on the water quality within the estuary. The most significant impact would be a permanent loss of about 40-acres of wetlands habitat and a temporary turbidity problem, both of which are associated with construction of dikes in the local protection project option. The third option, the Regional Floodgate Plan, has a minor impact on the water quality within the estuary.

5. The Regional Floodgate Plan which has as its major component, a floodgate located at the mouth of the Saugus River, affects water quality in the estuary in two ways: (1) by placing a constriction at the mouth, there is a change in velocities, scour, and sedimentation characteristics within a few hundred feet of the structure, and (2) the floodgates will be closed when the storm tides are expected to reach or exceed 8 feet NGVD to prevent flood damages, resulting in a temporary change in tidal flushing characteristics. This is expected to occur on the average about 2 to 3 times a year for a duration of about 1 to 2 hours around peak high tide for each operation. Normal flushing changes are expected to be negligible for an open gate condition.

6. With the proposed floodgate structure there are generally expected to be minimal changes in existing water quality conditions including salinity characteristics within the estuary since normal tide elevations, currents, and flushing volumes will show negligible differences from existing conditions and also since gate closures for storm tides will happen so infrequently.

7. Current speed and direction within a few hundred feet upstream and downstream of the selected floodgate structure are expected to change somewhat. Velocities in the center of the channel near the floodgate structure are expected to increase slightly over those of the existing condition; during peak flow for a mean spring tide condition, typically measured average velocities at the General Edwards bridge were 1.7 fps and calculated average velocities at the proposed structure would be 2.1 fps. Velocities near the riverbanks will reduce some and there may be a slight increase in sediment buildup generally near the southern side at the mouth of the river.

8. If the floodgate option continues into the project engineering and design phase, further current data and additional modeling is needed to define localized current velocity and direction and sedimentation/erosion characteristics within a few hundred feet of the structure. This is described in the Hydrology and Hydraulics Appendix, Addendum II.

SAUGUS RIVER AND TRIBUTARIES
FLOOD DAMAGE REDUCTION STUDY
LYNN, MALDEN, REVERE AND SAUGUS
MASSACHUSETTS

1. INTRODUCTION

a. Purpose. This report presents a description of water quality conditions as they presently exist in the Saugus and Pines Rivers estuary and as they would likely exist both with and without the proposed Regional Flood Reduction Project (see figure 1). Use was made of numerous water quality reports previously prepared by others who have studied the estuary as well as field data collected and analyzed by the Corps. Recommendations for further detailed study have also been presented should the project proceed into the project engineering and design phase (PED).

b. Project Alternatives. Three flood reduction alternatives have been evaluated for the study area. These alternatives are fully described in the feasibility report. A summarization is presented here.

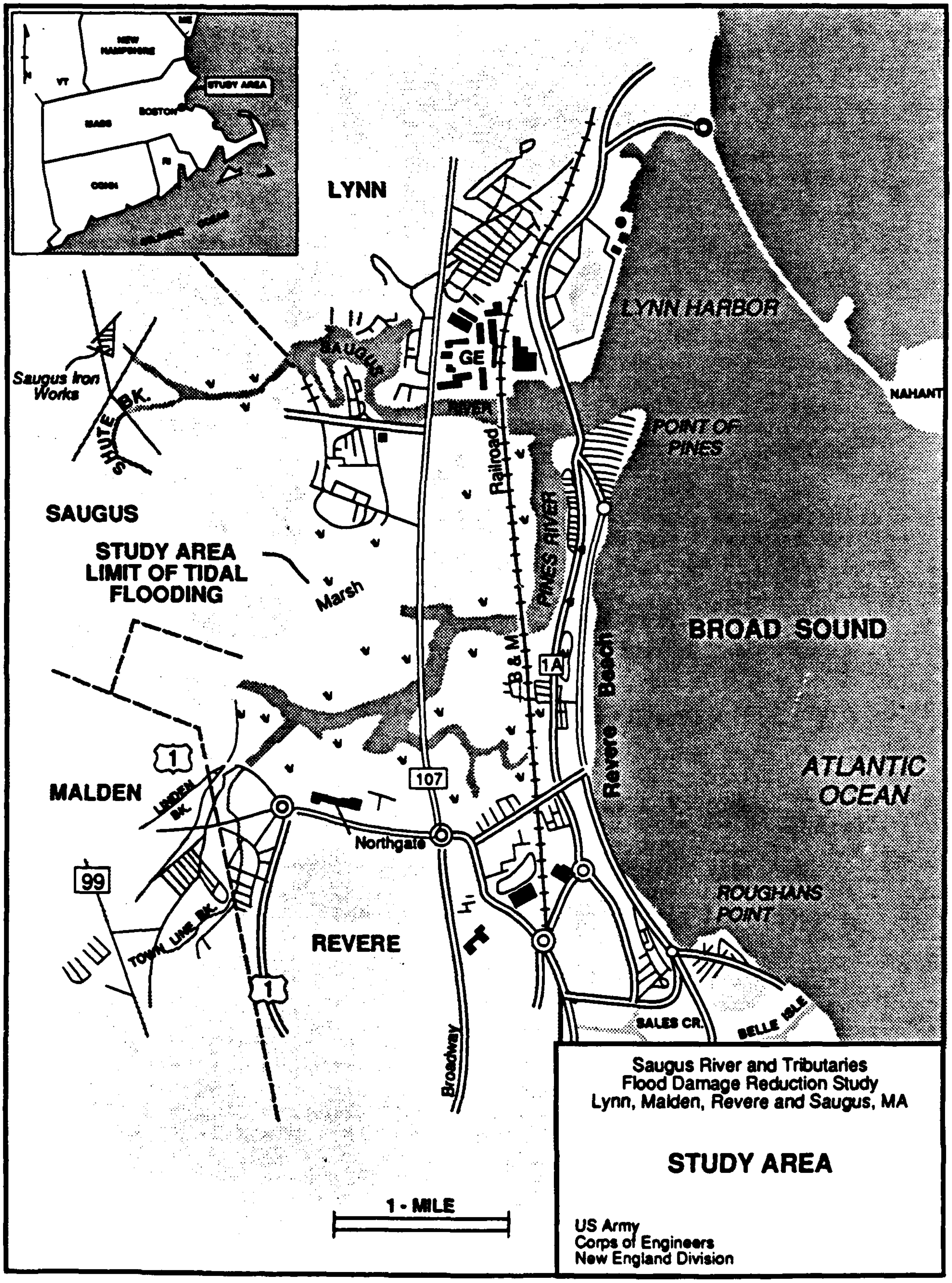
(1) Local Protection Plans. Consists of earth dikes or concrete walls located along the shorefront and riverbanks, aligned so as to protect the following areas:

Revere Beach Backshore
Town Line Brook
East Saugus
Lynn

Figure 2 shows the location of these local protection plans.

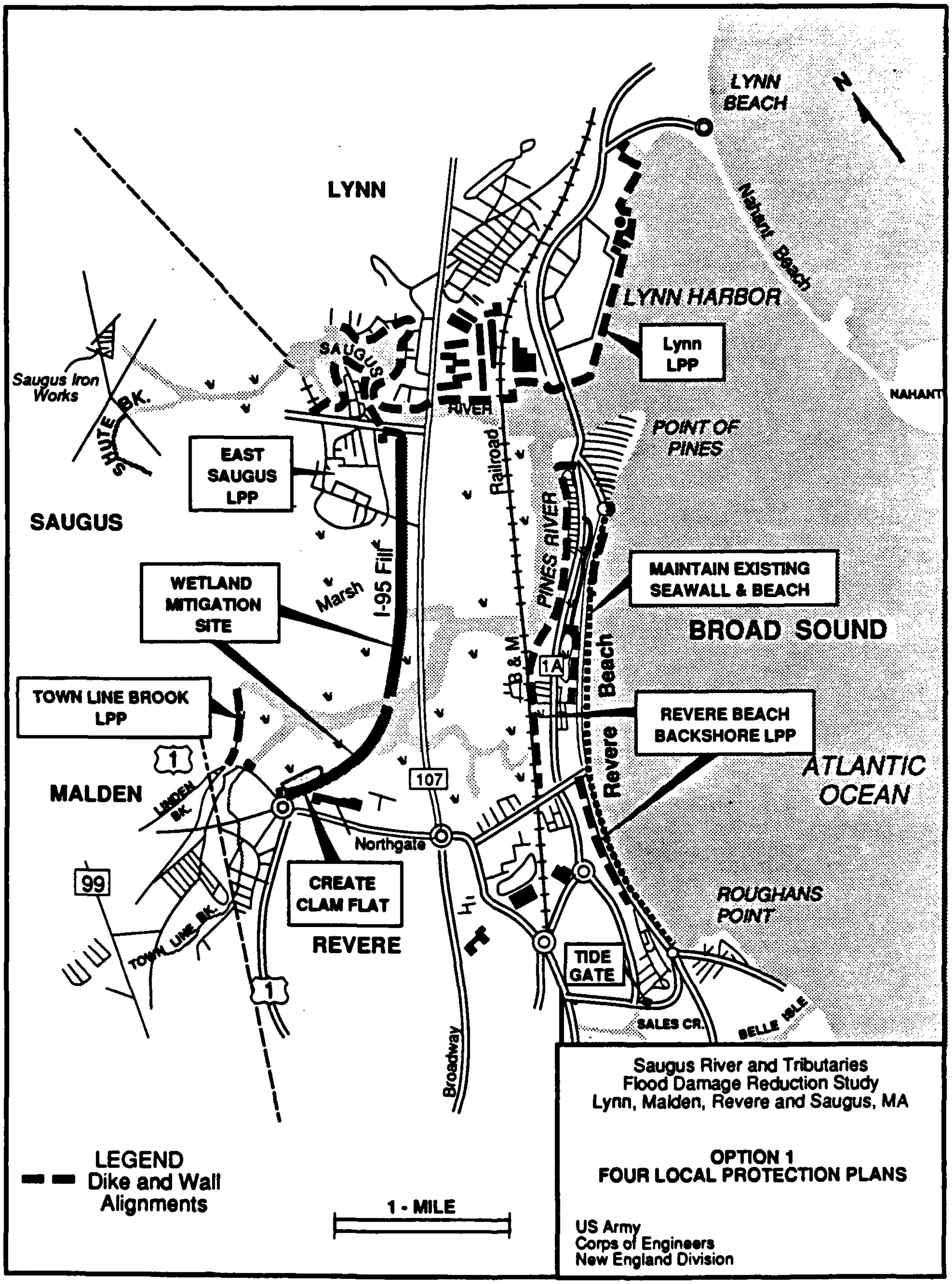
(2) Nonstructural Plans. Considered nonstructural measures to reduce flood damages through flood warning systems and flood proofing of individual buildings.

(3) Regional Floodgate Plan. Provides a tidal floodgate across the mouth of the Saugus River along with a shorefront dike and wall system which ties into high ground (figure 3). Primary emphasis in this report is given to this plan since at present it appears to be the most likely recommended alternative. Also it raises the greatest interest for possible water quality impacts. The gated structure, as presently envisioned, consists of one 100-foot wide navigation gate, extending to -13.4 MLW (-18.0 feet NGVD) and ten 50-foot wide flushing gates with inverts (bottom of openings) at -9.4 MLW (-14.0 feet NGVD) and soffit (top of opening) levels of 0 feet NGVD (for detailed plan and



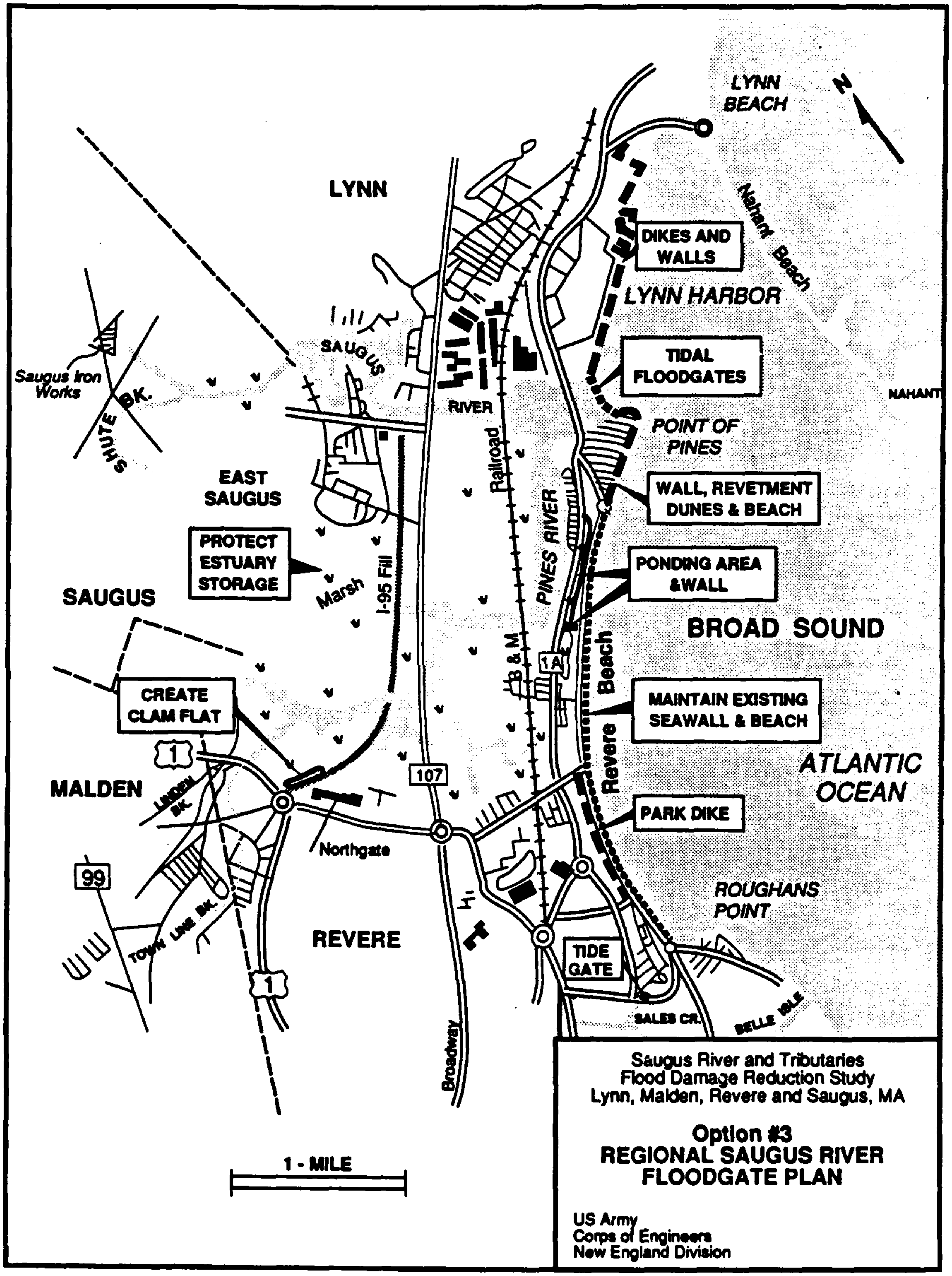
LYNN
LYNN HARBOR
NAHANT
GE
SAUGUS RIVER
Railroad
POINT OF PINES
Saugus Iron Works
SHUTE BK.
SAUGUS
STUDY AREA LIMIT OF TIDAL FLOODING
Marsh
PINES RIVER
BROAD SOUND
B & M
1A
Revere Beach
ATLANTIC OCEAN
1
MALDEN
LINDEN BK.
107
Northgate
99
TOWN LINE BK.
REVERE
ROUGHANS POINT
SALES CR.
BELLE ISLE
Broadway
1 - MILE
Saugus River and Tributaries
Flood Damage Reduction Study
Lynn, Malden, Revere and Saugus, MA
STUDY AREA
US Army
Corps of Engineers
New England Division
NEW HAMPSHIRE
VT
STUDY AREA
BOSTON
MASS
CONN
RI

FIGURE 1



LYNN BEACH
N
LYNN
Nahant Beach
LYNN HARBOR
Lynn LPP
NAHANT
Saugus Iron Works
SHUTE BK.
SAUGUS RIVER
EAST SAUGUS LPP
POINT OF PINES
Railroad
SAUGUS
PINES RIVER
I-95 Fill
Marsh
WETLAND MITIGATION SITE
MAINTAIN EXISTING SEAWALL & BEACH
BROAD SOUND
B & M
1A
Revere Beach
TOWN LINE BROOK LPP
REVERE BEACH BACKSHORE LPP
1
ATLANTIC OCEAN
MALDEN
LINDEN BK.
107
Northgate
99
CREATE CLAM FLAT
ROUGHANS POINT
REVERE
TOWN LINE BK.
TIDE GATE
1
SALES CR.
BELLE ISLE
Broadway
Saugus River and Tributaries
Flood Damage Reduction Study
Lynn, Malden, Revere and Saugus, MA
OPTION 1
FOUR LOCAL PROTECTION PLANS
US Army
Corps of Engineers
New England Division
LEGEND
Dike and Wall Alignments
1 - MILE

FIGURE 2



LYNN BEACH
N
LYNN
Nahant Beach
DIKES AND WALLS
LYNN HARBOR
TIDAL FLOODGATES
NAHANT
SAUGUS
RIVER
Saugus Iron Works
SHUTE BK.
POINT OF PINES
Railroad
EAST SAUGUS
WALL, REVETMENT DUNES & BEACH
PINES RIVER
PROTECT ESTUARY STORAGE
Marsh
I-95 Fill
PONDING AREA &WALL
SAUGUS
BROAD SOUND
B & M
1A
Beach
MAINTAIN EXISTING SEAWALL & BEACH
CREATE CLAM FLAT
1
107
Revere
ATLANTIC OCEAN
MALDEN
LINDEN BK.
PARK DIKE
Northgate
99
ROUGHANS POINT
TOWN LINE BK.
REVERE
TIDE GATE
1
SALES CR.
BELLE ISLE
Broadway
1 - MILE
Saugus River and Tributaries
Flood Damage Reduction Study
Lynn, Malden, Revere and Saugus, MA
Option #3
REGIONAL SAUGUS RIVER
FLOODGATE PLAN
US Army
Corps of Engineers
New England Division

FIGURE 3

section view, see Main Report). Total flow area at mean spring tide level (5.7 feet NGVD) through the structure is 9,170 square feet (SF) as compared to the total estimated existing flow area of the channel at the General Edwards bridge of 15,540 SF. At the level of peak or maximum flow (about 0.0 feet NGVD) the gated flow area of 8,800 SF is nearly equal to the existing riverflow area (8,700 SF) at the location of the floodgates. For this report, the floodgate structure was assumed to be located on the Saugus River about 700 feet downstream (east) from the General Edwards bridge (State Route 1A). Several nearby alignments were considered by planners and are described in the Main Report. The flushing gates were assumed to be distributed across the width of the channel with nine gates located north of the navigation channel and one gate located south of the channel. Actual project location and gate sizes are subject to change as more refined engineering analysis and model studies are completed. The present layout should be viewed as a concept useful in assessing relative impacts. Also, when the floodgate is described as being opened or closed in this report, it is assumed that all gates, both navigation and flushing, are opened or closed.

Navigation and flushing gates will remain fully open at all times, except when storm tide levels are predicted to be greater than those expected to produce flood damage. Until more detailed damage-tide elevation relationships are determined around the entire tidal estuary, the beginning of flood damages is estimated to be about 8 feet NGVD.

During flood operation of the tidal floodgate, the gates will generally be closed for only a few hours around the time of high predicted tide and will be opened as soon as the ocean level drops to the level of the estuary. The gates will be closed on the average of 2 to 3 times a year. More detailed information on the floodgate operation is given in the Hydrology and Hydraulics Appendix.

c. Other Projects and Studies within the Saugus and Pines Estuary. There are a number of potential future conditions which could result in increases in the existing flushing characteristics of the estuary (see Main Report); among these are sea level rise, possible but unlikely breaching or removal of the abandoned I-95 highway embankment and proposed channel dredging for the Corps Saugus River navigation project. Although the sensitivity of tidal flushing was evaluated if the I-95 embankment was removed, it has been assumed for this study that it would remain in place. Under both proposed flood protection and no action alternatives, it is assumed that the Saugus River navigation dredging project will be constructed. These conditions or projects are fully described in the feasibility report.

In addition, there are a number of ongoing studies which will be addressing water quality conditions in the estuary, e.g., Lynn Water and Sewer Commission's (LWSC) combined sewer overflow study and Refuse Energy Systems Company's (RESCO) landfill capping study. Both of these investigations are yet to be completed. Preliminary indications are that the combined sewer overflow being discharged into the Saugus River will be eliminated as the combined sewer portion is separated. The landfill capping study is currently identifying problems to be eliminated. Both recommendations result in reduction of pollutional loads to the estuary.

Although there are no present plans to dredge the Pines River, there is local interest which may eventually result in dredging this river for navigation.

d. Watershed Description. The Saugus River forms at Lake Quannapowitt in Wakefield and follows a meandering southeasterly course through flat to rolling topography for 13 miles to its mouth in Lynn Harbor. The drainage pattern consists of a series of wetland areas interconnected by a system of streamlike channels. The Pines River forms at the junction of Town Line and Linden Brooks in northeastern Revere, flows easterly for 2.5 miles and then northerly for 1.2 miles where it joins the Saugus River about 0.4 mile upstream from its mouth. Extensive vegetated wetlands (approximately 1,070 acres) border the lower 4.5 miles of the Saugus River and the entire length of the Pines River. Total drainage area is 47 square miles (see plate 1). There is no USGS streamflow gage located on the river; however, as described in the Hydrology and Hydraulics Appendix, estimated average annual flow is equal to 80 cfs (23 inches of runoff/ year). The USGS has installed several tide gaging stations throughout the study area.

The entire river system is surrounded by dense urban development which results in numerous occurrences of nonpoint pollutant discharges within the basin. There are no major continuous point source pollutant discharges within the basin, although there are several major thermal water releases made by industries and an intermittent discharge from a combined sewer overflow. In general, most pollutants are diluted as a result of the large tidal interchange which exists in the estuary: mean tidal range is 9.5 feet and mean spring tidal range is 11.0 feet. Pollutants have also been assimilated into the salt marsh muds through sedimentation, adsorption, and absorption processes.

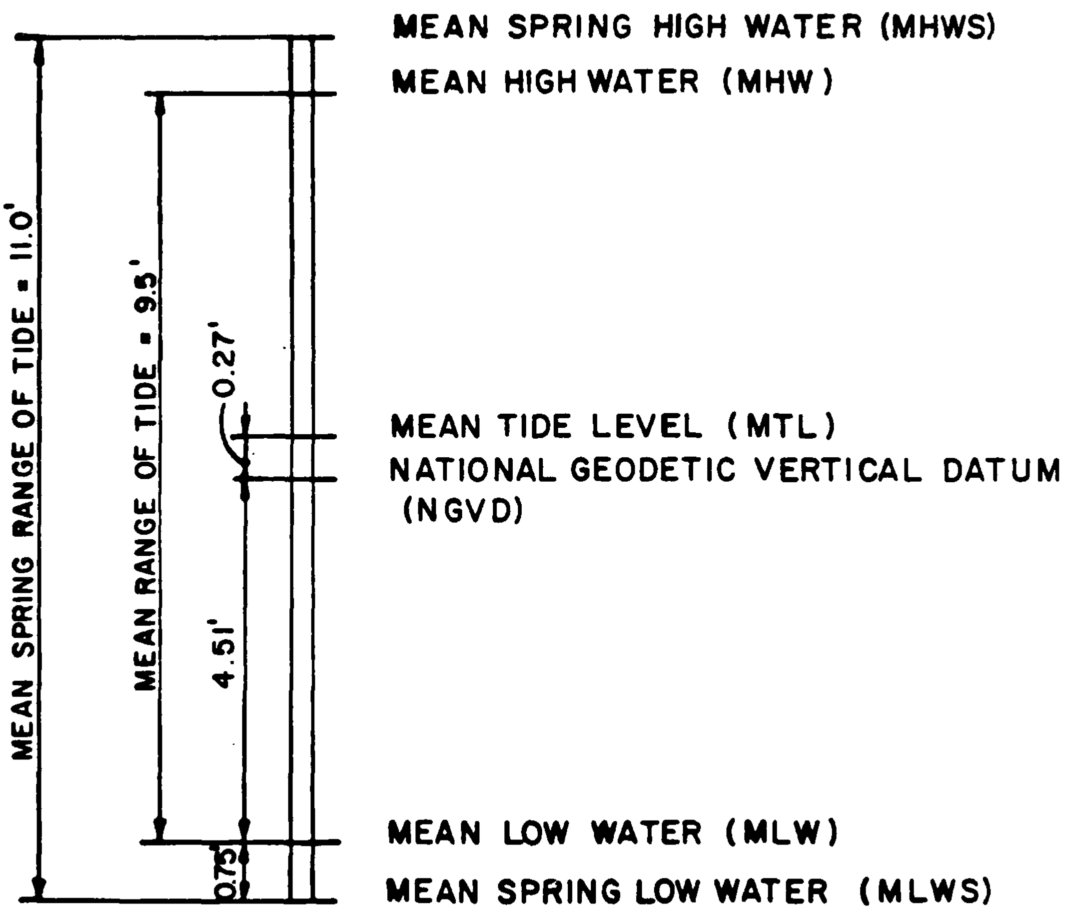
e. Normal Tides

(1) General. At the study area, tides are semidiurnal, with two high and two low waters occurring during each lunar day (approximately 24-hours 50-minutes). The resulting tide range is constantly varying in response to the relative positions of the earth, moon, and sun; the moon having the primary tide producing effect. Maximum tide ranges occur when the orbital cycles of these bodies are in phase. A complete sequence of tide ranges is approximately repeated over an interval of 19 years, which is known as a tidal epoch. At the National Ocean Survey (NOS) tide gage in Boston, Massachusetts (less than 10 miles from the study site), the mean range of tide and the mean spring range of tide are 9.5 and 11.0 feet, respectively. The maximum and minimum predicted astronomic tide ranges at Boston have been estimated at about 14.7 and 5.0 feet, respectively, using the Corps Coastal Engineering Research Center (CERC) report entitled: "Tides and Tidal Datums in the United States," SR No. 7, 1981.

Because of the continual variation in water level due to the tides, several reference planes, called tidal datums, have been defined to serve as a reference zero for measuring elevations of both land and water. Tidal datum information for Boston is presented in figure 4 and table 1. This data was compiled using currently available NOS tidal benchmark data for Boston along with the previously mentioned CERC report. The epoch for which the NOS has published tidal datum information for Boston is 1960-78. A phenomenon that has been observed through tide gaging and tidal benchmark measurements is that sea level is apparently rising with respect to the land along most of the U.S. coast. At the Boston National Ocean Survey tide gage, the rise has been observed to be slightly less than 0.1 foot per decade. Sea level determination is generally revised at intervals of about 25 years to account for the changing sea level phenomenon (sea level rise is further discussed in the Hydrology and Hydraulics Appendix).

TIDAL DATUM PLANES
BOSTON, MASSACHUSETTS
NATIONAL OCEAN SURVEY TIDE GAGE

(BASED UPON 1960-78 NOS TIDAL EPOCH)



NEW ENGLAND DIVISION
U.S. ARMY, CORPS OF ENGINEERS
WALTHAM, MASS. MARCH 1985

FIGURE 4

TABLE 1

BOSTON TIDAL DATUM PLANES
NATIONAL OCEAN SURVEY TIDE GAGE
(Based Upon 1960-78 NOS Tidal Epoch)

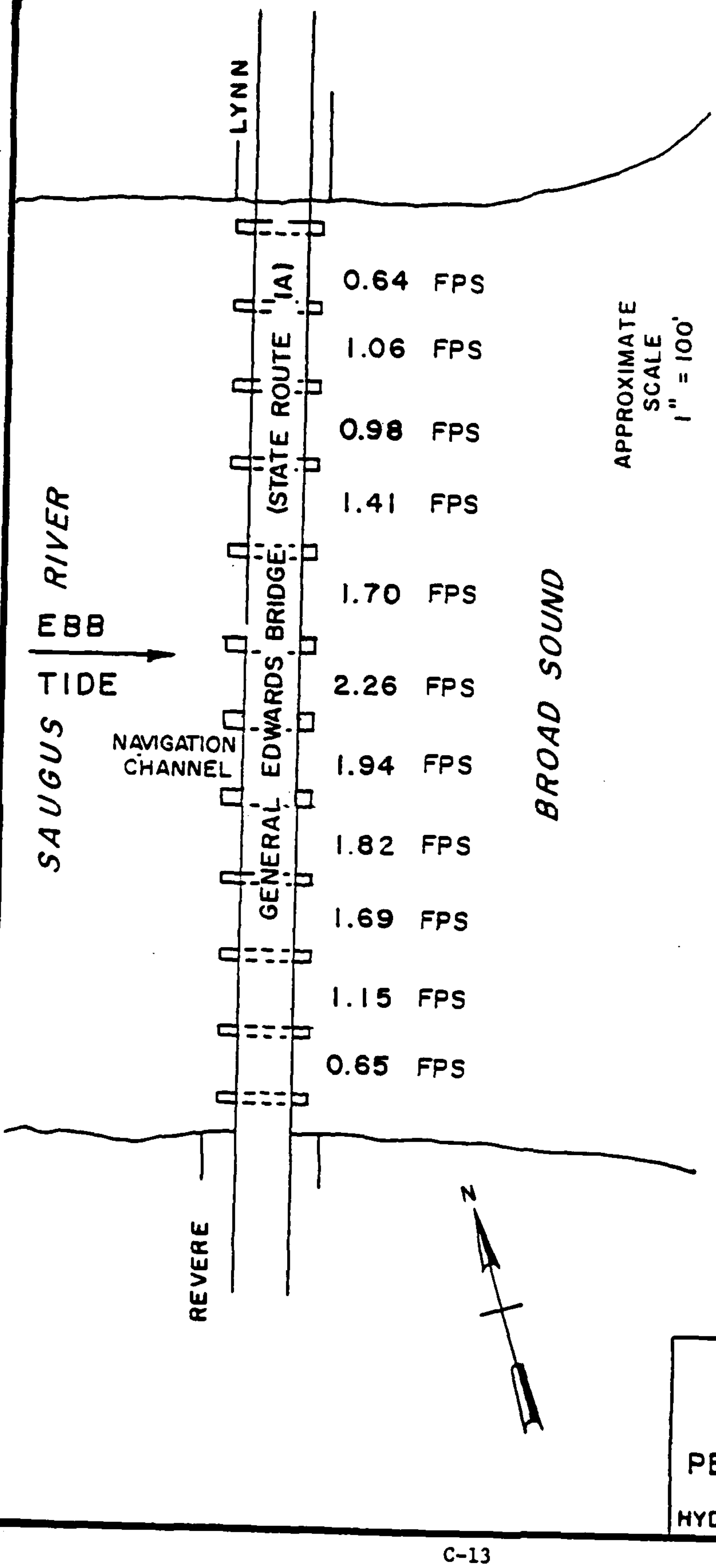
	Tide Level (ft, NGVD)
Maximum Predicted Astronomic High Water	7.5
Mean Spring High Water (MSHW)	5.8
Mean High Water (MHW)	5.0
Minimum Predicted Astronomic High Water	2.7
Mean Tide Level (MTL)	0.3
National Geodetic Vertical Datum (NGVD)	0.0
Maximum Predicted Astronomic Low Water	-2.4
Mean Low Water (MLW)	-4.5
Mean Spring Low Water (MLWS)	-5.2
Minimum Predicted Astronomic Low Water	-7.1

(2) Tidal Fluctuation in the Saugus and Pines Rivers. Due to the complexity of water movement within the Saugus and Pines Rivers estuary, tidal stage measurements have been made intermittently over the past three years to better define tidal motion within the project area. In general, tide levels at the mouth of the Saugus River are found nearly identical to those at Boston. Tide levels in the estuary show some slight variance. In the Saugus River, at the Lincoln Street bridge, and in the upper Pines River within the Seaplane Basin, high water levels are 0.1 and 0.5 foot lower at mean spring tide than at the mouth, respectively. Low water levels are about the same and 0.3 foot higher at mean low tide, respectively. Plate 2 shows the location of tidal staff and recording gages for the Corps investigation. More detailed data is tabulated in the Hydrology and Hydraulics Appendix.

(3) Tidally Induced Currents. The Corps in cooperation with the U.S. Geological Survey conducted an investigation to determine the tidal volume exchanged during a normal tide cycle on 16 April 1987. Current measurements were made over the course of a full tide cycle (from low to high to low) on the downstream side of the General Edwards bridge (State Route 1A). Tide levels ranged from approximately -5.5 to +5.6 feet to -4.4 feet NGVD during the gaging period, and peak current measurements at the center of the bridge were 2.44 fps during flood and ebb tides. The average velocity during maximum discharge was 1.74 fps. Plan views of the currents as measured across the width of the channel during peak flow condition for both a flood and ebb condition are shown in figures 5 and 6, respectively.

To further define the characteristics of the estuary, concurrent cross-sectional areas at the bridge were collected with the velocities on 16 April so that tidal interchange volumes could be estimated for that day. On 16 April, between 7:30 a.m. and 8 p.m., approximately 5,800 acre-feet of water flowed into the estuary and approximately 6,300 acre-feet flowed out of the estuary (a typical mean spring tide condition). The difference in volumes was assumed to be the result of interior runoff as well as a dramatic change in wind direction which occurred about mid high tide on that day. High onshore winds tend to pile up water in the estuary. The winds may have impeded tides from flowing out during the previous tide cycle. At high tide on this particular day, the tidally influenced water surface area upstream from the General Edwards bridge was estimated to be 1,000 acres.

In addition to the data collected on 16 April, one-time current measurements were also made during a midtide flood condition at several bridge openings within the basin on 20 April 1987 when the tide ranged between -4.9 and +4.0 feet NGVD. Currents were measured at the Fox Hill drawbridge (Rte 107) and the Lincoln Avenue bridge over the Saugus River, and the State Route 107 bridge over the Pines River. Representative peak midtide flood velocities measured at each bridge were 1.4, 2.6 and 3.0 fps, respectively. Measurements were made at these bridges to provide background data for these restrictions, some of the narrowest river restrictions which exist within the estuary. All current data is fully described in the Hydrology and Hydraulics Appendix. A map showing the location of current measurements is shown in figure 7.



NOTES:

CURRENTS MEASURED AT CENTERLINE OF EACH SPAN BETWEEN PIERS.

HIGHER VELOCITIES WERE MEASURED AT CERTAIN SPANS, HOWEVER, THE VELOCITY AT THIS INSTANT PRODUCED THE LARGEST TOTAL DISCHARGE.

1 KNOT = 1.689 fps

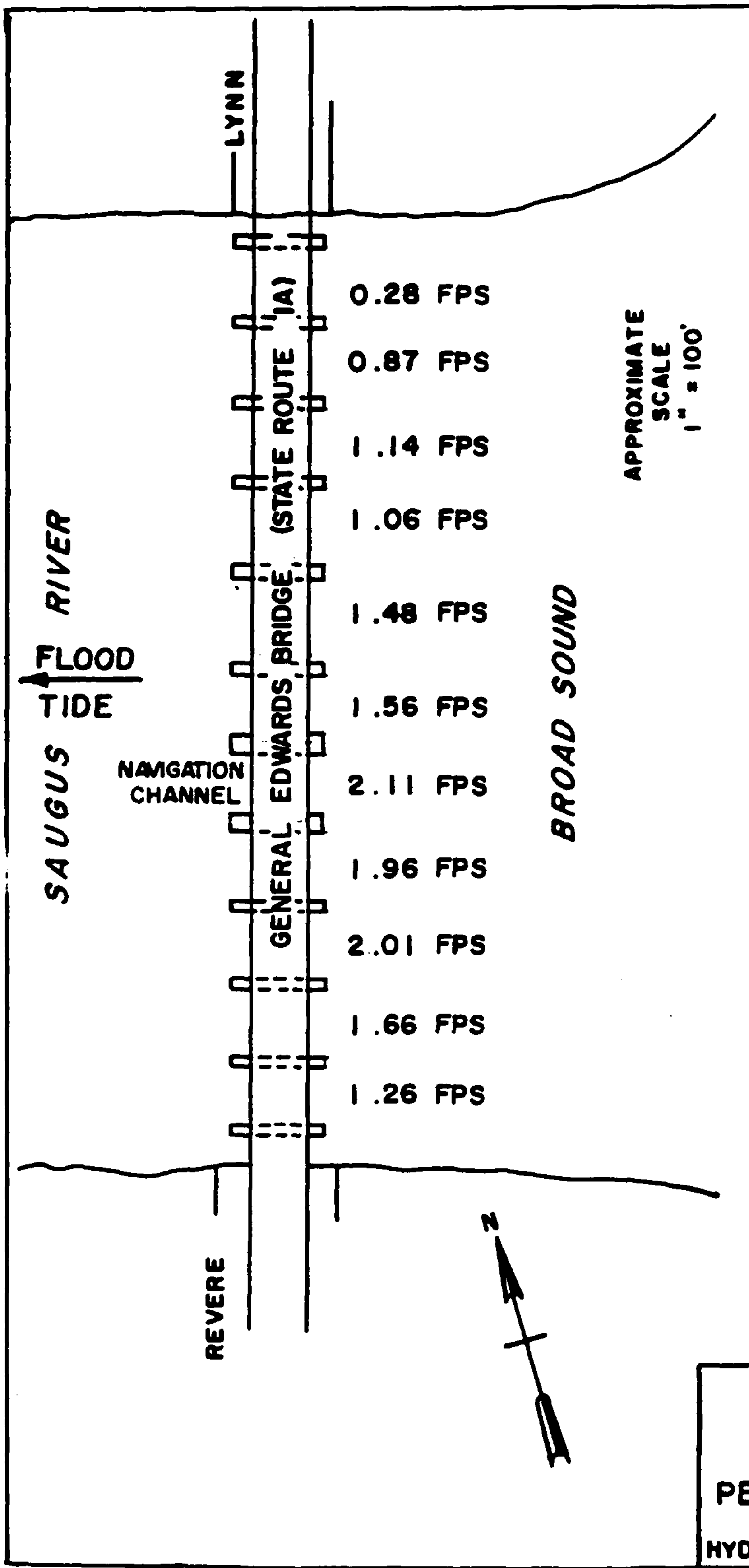
RIVER CURRENTS AS MEASURED ON APRIL 16, 1987
TIME: 4:30 PM

MID TIDE CURRENTS

PEAK EBB CONDITION

HYD. & W.Q. SECT. JUNE 1988

FIGURE 5



NOTES:

CURRENTS MEASURED AT CENTERLINE OF EACH SPAN BETWEEN PIERS.

HIGHER VELOCITIES WERE MEASURED AT CERTAIN SPANS, HOWEVER, THE VELOCITY AT THIS INSTANT PRODUCED THE LARGEST TOTAL DISCHARGE.

1 KNOT = 1.689 fps

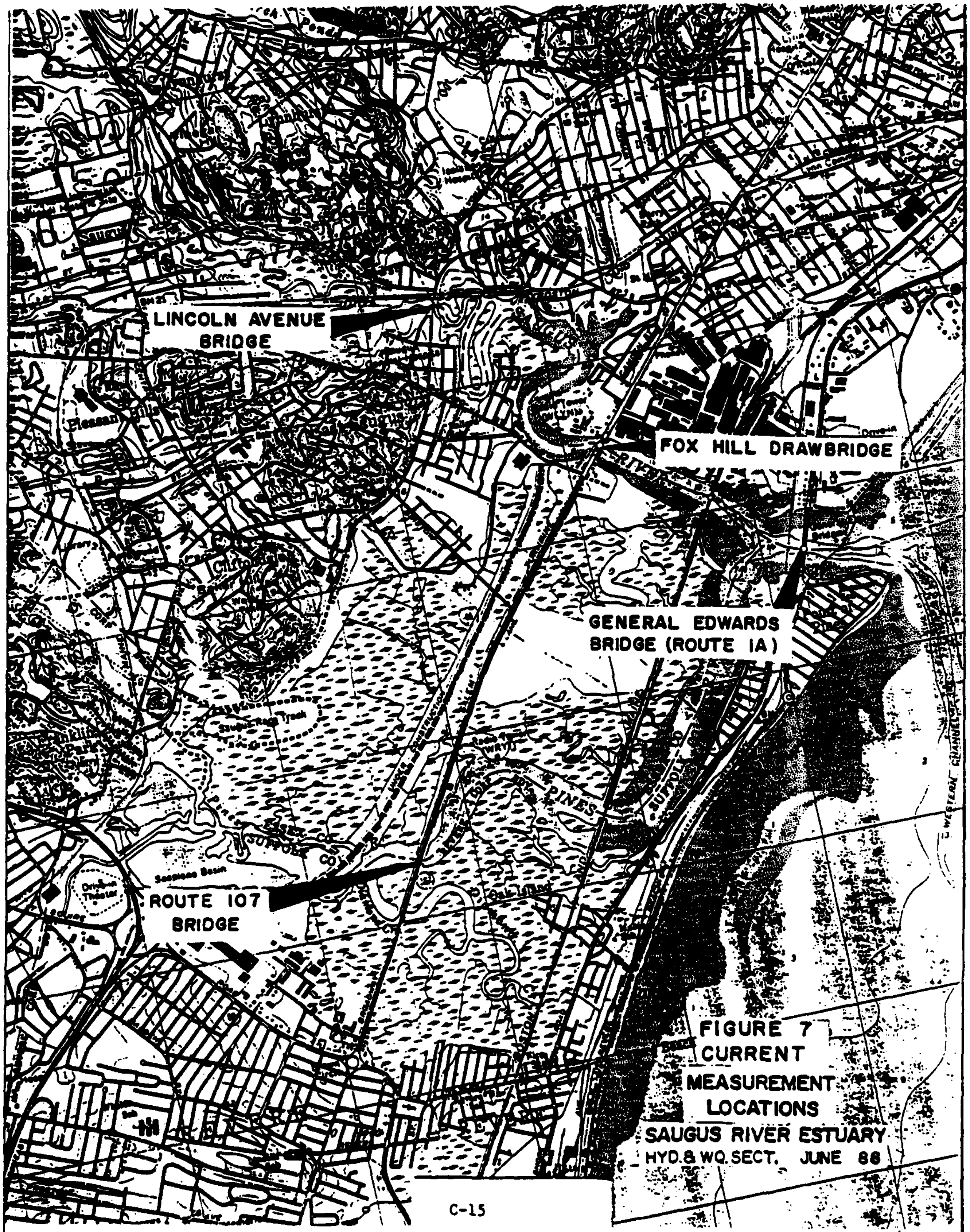
RIVER CURRENTS AS MEASURED ON APRIL 16, 1987
TIME: 11:30 AM

MID TIDE CURRENTS

PEAK FLOOD CONDITION

HYD. & W.Q. SECT. JUNE 1988

FIGURE 6

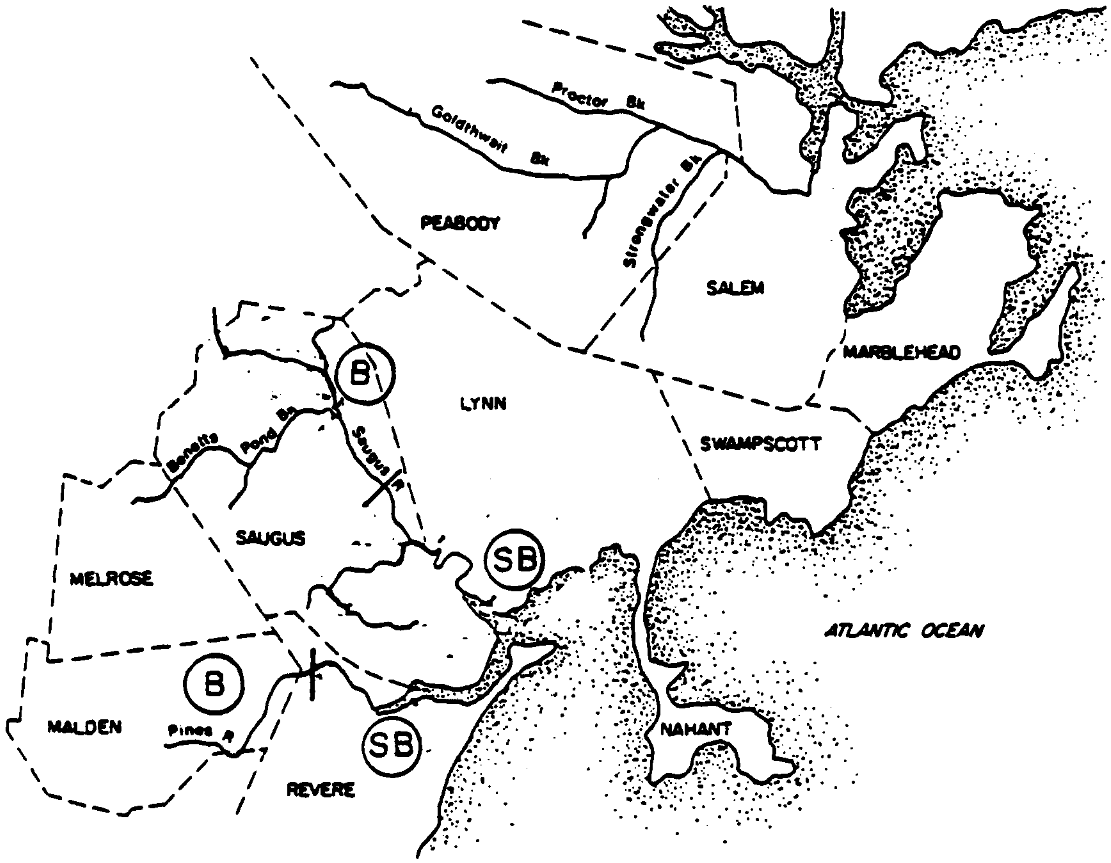

LINCOLN AVENUE BRIDGE
FOX HILL DRAWBRIDGE
GENERAL EDWARDS BRIDGE (ROUTE 1A)
ROUTE 107 BRIDGE
FIGURE 7
CURRENT MEASUREMENT LOCATIONS
SAUGUS RIVER ESTUARY
HYD.& WQ. SECT. JUNE 88

2. EXISTING WATER QUALITY

a. Water Quality Classification. The inland waters of the Saugus and Pines Rivers have been designated class B and the coastal waters of these rivers have been designated class SB by the Massachusetts Division of Water Pollution Control (MDWPC). The change from class SB to class B occurs at the Metropolitan District Commission's (MDC) tide gate near U.S. Route 1 on the Pines River and just upstream from the Saugus Ironworks on the Saugus River (approximately 4.5 miles upstream from the mouth) (see figure 8). Class B waters are suitable for swimming, other recreation, and for protection and propagation of fish, other aquatic life and wildlife. Class SB waters, in addition to those designated uses as described for class B waters, are suitable for shellfish harvesting with depuration.

Massachusetts Surface Water Quality Standards (1985) for class B waters include a minimum dissolved oxygen (DO) level of 5 mg/l, a maximum water temperature of 83° Fahrenheit, pH between 6.5 to 8.0 Standard Units (SU), or as naturally occurring, and fecal coliform counts not to exceed a median value of 200 colonies/100 ml nor more than 400 colonies/100 ml for any monthly sampling period. For class SB waters these standards include a minimum of 6.0 mg/l of dissolved oxygen at water temperatures of 77° Fahrenheit and below, no increase in temperatures which will exceed the recommended limits for the most sensitive water use, pH in the range of 6.5 to 8.5 or as naturally occurring, and total coliform counts not to exceed a median value of 700 colonies/100 ml nor more than 1,000 colonies/100 ml for 20 percent of the samples. Shellfishing is the most sensitive activity in the coastal area due to the stringent requirements established to prevent contamination of clams and other bivalves harvested for human consumption. Classification of shellfish areas are additionally subdivided into three categories: (1) clean (open) - coliform bacteria counts between 0 to 70 colonies/ 100 ml of water, (2) moderately contaminated (restricted) - between 71 to 100 colonies/100 ml, and (3) grossly contaminated (closed) - over 700 colonies/ 100 ml.

The complete technical requirement for the Massachusetts water classification for B and SB waters is given in table 2. The classification is the goal for a particular water body and not necessarily a description of existing conditions.



N

Not to scale

LEGEND

MASSACHUSETTS W.Q. CLASSIFICATION (B)

SAUGUS RIVER BASIN

WATER QUALITY CLASSIFICATION

HYD. & W.Q. SECT. JUNE 1988

FIGURE 8

TABLE 2

MASSACHUSETTS
WATER QUALITY CLASSIFICATION CRITERIA
PINES AND SAUGUS RIVERS

Class B - Waters assigned to this class are designated for the uses of protection and propagation of fish, other aquatic life and wildlife, and for primary and secondary contact recreation.

Parameter	Criteria
Dissolved Oxygen	Shall be a minimum of 5.0 mg/l in warm water fisheries and a minimum of 6.0 mg/l in cold water fisheries.
Temperature	Shall not exceed 83° F (28.3° C) in warm water fisheries or 68° F (20° C) in cold water fisheries, nor shall the rise resulting from artificial origin exceed 4° F (2.2° C).
pH	Shall be in the range of 6.5 to 8.0 standard units and not more than 0.2 unit outside of the naturally occurring range.
Fecal Coliform Bacteria	Shall not exceed a log mean for a set of samples of 200 per 100 ml, nor shall more than 10 percent of the total samples exceed 400 per 100 ml during any monthly sampling period.
Aesthetics	All waters shall be free from pollutants in concentrations or combinations that: a. Settle to form objectionable deposits. b. Float as debris, scum or other matter to form nuisances. c. Produce objectionable odor, color, taste or turbidity. d. Result in the dominance of nuisance species.

Radioactive Substances	Shall not exceed the recommended limits of the U.S. Environmental Protection Agency's National Drinking Water Regulations.
Tainting Substances	Shall not be in concentrations or combinations that produce undesirable flavors in the edible portions of aquatic organisms.
Color, Turbidity, Total Suspended Solids	Shall not be in concentrations or combinations that would exceed the recommended limits on the most sensitive receiving water use.
Oil and Grease	The water surface shall be free from floating oils, greases and petrochemicals and any concentrations in the water column or sediments that are aesthetically objectionable or deleterious to the biota are prohibited. For oil and grease of petroleum origin the maximum allowable discharge concentration is 15 mg/l.
Nutrients	Shall not exceed the site-specific limits necessary to control accelerated or cultural eutrophication.
Other Constituents	Waters shall be free from pollutants in concentrations or combinations that: a. Exceed the recommended limits on the most sensitive receiving water use. b. Injure, are toxic to, or produce adverse physiological or behavioral responses in humans or aquatic life. c. Exceed site-specific safe exposure levels determined by bioassay using sensitive species.

Class SB - Waters assigned to this class are designated for the uses of protection and propagation of fish, other aquatic life and wildlife; for primary and secondary contact recreation; and for shellfish harvesting with depuration (restricted shellfish areas).

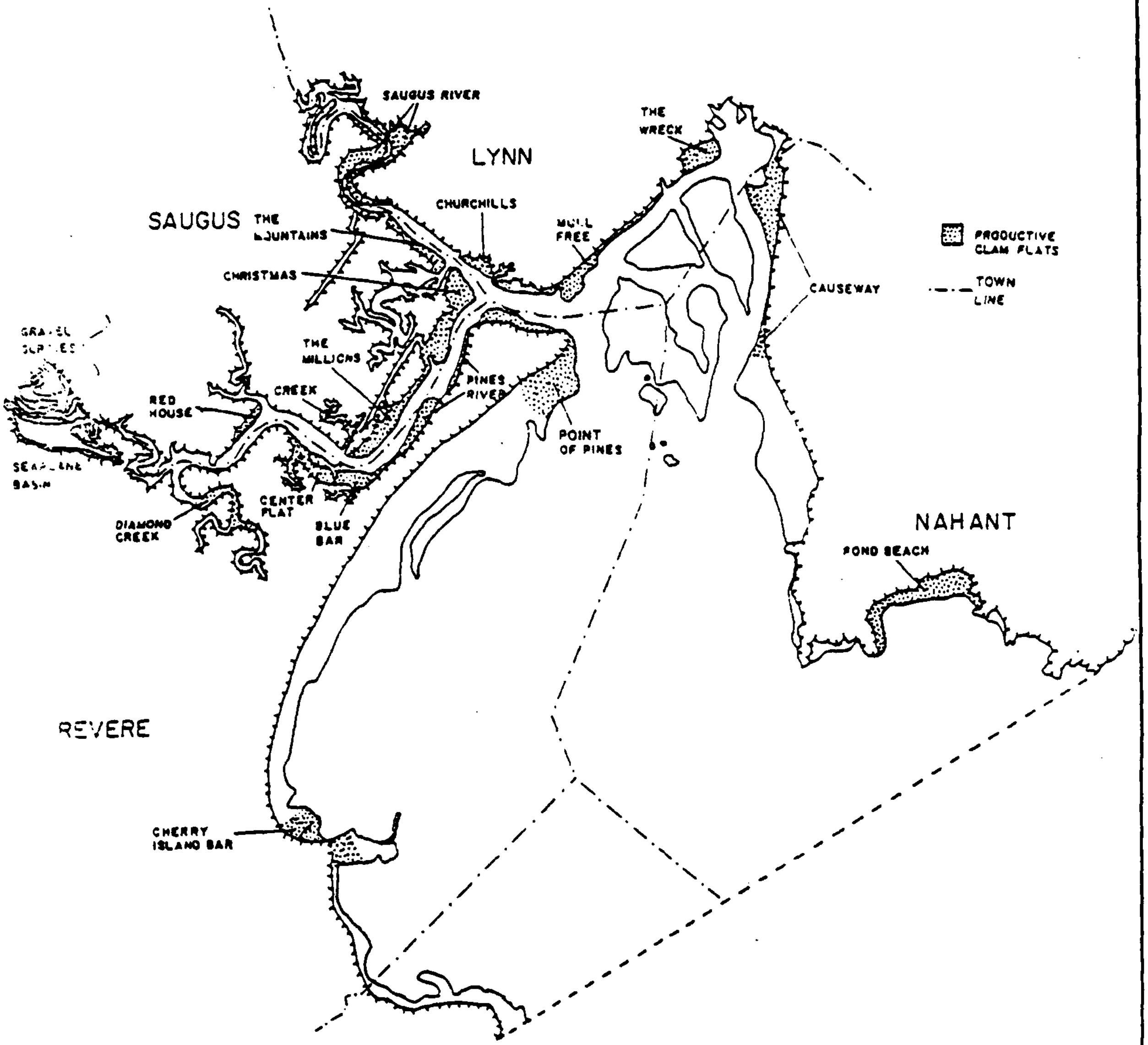
Parameter	Criteria
Dissolved Oxygen	Shall be minimum of 85 percent of saturation at water temperatures above 77° F (25° C) and shall be a minimum of 6.0 mg/l at water temperatures of 77° F (25° C) and below.
Temperature Increase	None except where the increase will not exceed the recommended limits on the most sensitive water use.
pH	Shall be in the range of 6.5 to 8.5 and not more than 0.2 unit outside of the naturally occurring range.
Total Coliform Bacteria	Shall not exceed a median value of 700 MPN per 100 ml and not more than 20 percent of the samples shall exceed 1,000 MPN per 100 ml during any monthly sampling period.
Aesthetics	All waters shall be free from pollutants in concentrations or combinations that: a. Settle to form objectionable deposits. b. Float as debris, scum or other matter to form nuisances. c. Produce objectionable odor, color, taste or turbidity. d. Result in the dominance of nuisance species.
Radioactive Substance	Shall not exceed the recommended limits of the U.S. Environmental Protection Agency's National Drinking Water Regulations.

Tainting Substances	Shall not be in concentrations or combinations that produce undesirable flavors in the edible portions of aquatic organisms.
Color, Turbidity, Total Suspended Solids	Shall not be in concentrations or combinations that would exceed the recommended limits on the most sensitive receiving water use.
Oil and Grease	The water surface shall be free from floating oils, grease and petrochemicals and any concentrations or combinations in the water column or sediments that are aesthetically objectionable or deleterious to the biota are prohibited. For oil and grease of petroleum origin the maximum allowable discharge concentration is 15 mg/l.
Nutrients	Shall not exceed the site-specific limits necessary to control accelerated or cultural eutrophication.
Other Constituents	Waters shall be free from pollutants in concentration or combinations that: a. Exceed the recommended limits on the most sensitive receiving water use. b. Injure, are toxic to, or produce adverse physiological or behavioral responses in human aquatic life. c. Exceed site-specific safe exposure levels determined by bioassay using sensitive species.

b. Water Quality Conditions. According to the "Saugus River Basin Water Quality Survey" prepared by the Massachusetts Division of Water Pollution Control (MDWPC) in November 1982, the Saugus and Pines Rivers generally meet class B and SB standards during dry weather flow with a few minor violations occurring from high coliform counts caused by combined sewers, illegal sewer connections, and failing septic systems. During storm events; however, discharges from storm drains and overland flow have a significant adverse impact on the quality in the upper estuary above Route 107, principally because these discharges make up such a large percentage of the volume. Dissolved oxygen levels are impacted due to high quantities of BOD discharged (estimated at 17 times normal levels for one storm event measured during the Lynn Water and Sewer Commission's study of combined sewer overflows). Coliform levels are extremely high; measurements were as high as 30,000 colonies/100 ml for one storm. In the lower basin, the BOD levels do not have as severe an impact due to the large volume interchange which occurs as a result of tidal action. Coliform levels are significant enough; however, that standards are consistently exceeded even in the downstream area during high runoff events.

Because of the high coliform levels in the estuary, the mudflats have not been classified open for shellfish harvesting in recent years. A few areas have been classified as restricted; whereby, licensed master diggers and their employees may harvest shellfish and then have them depurated at the shellfish purification plant in Newburyport, MA. The most recent areas that have been designated restricted are in the upper Pines River; Gravel Gurties Sand Bar located in the Seaplane Basin and Center Bar located north of Oak Island. Locations of these areas are shown in figure 9. Coliform levels are monitored closely by the Department of Environmental Quality Engineering's (DEQE) Division of Shellfishing.

Results of the 1982 MDWPC testing for cadmium, chromium, mercury and zinc show that concentrations in the lower estuary downstream from the Route 107 bridges, generally meet the latest Quality Criteria for Water (1986) established by EPA. From the metals tested, cadmium appears to be the only one which approaches the limit established for protection of sensitive aquatic species in the marine environment (levels were measured at 40 mg/l for one grab sample). Table 3 shows the values of the MDWPC data for tidal areas and EPA criteria for water quality and figure 10 shows the locations where samples were taken.



NOTE:

MAP TAKEN FROM "A STUDY OF MARINE RESOURCES" BY MASS. DIVISION OF FISHERIES.

LOCATION OF PRODUCTIVE SOFT SHELL CLAM HABITAT

SAUGUS RIVER ESTUARY

JUNE 1988

FIGURE 9

LOCATION OF
MASSACHUSETTS DWPC
1982 WATER QUALITY
SAMPLING STATIONS
SAUGUS RIVER BASIN
JUNE 1988

FIGURE 10

TABLE 3

MDWPC
SAUGUS RIVER BASIN SURVEY
METALS DATA
(in mg/l)

DATE OF SAMPLE	20 JULY 1982				15 SEPTEMBER 1982				EPA CRITERIA	
STATION:	SR01	SR02	SR03	PR04	SR01	SR02	SR03	PR04	MARINE ACUTE CRITER.	MARINE CHRONIC CRITER.
Aluminum	0.18	0.18	0.20	0.18	0.22	0.28	0.34	0.31	-	-
Cadmium	0.01	0.01	0.01	0.01	0.04	0.04	0.04	0.04	0.043	0.093
Chromium, Total	0.06	0.07	0.07	0.06	0.02	0.04	0.03	0.03	10.3	-
Chromium, Hexavalent	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.1	0.05
Iron	0.38	0.40	0.38	0.38	0.20	0.22	0.24	0.24	-	-
Mercury	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0021	0.000025
Zinc	0.03	0.02	0.04	0.03	0.03	0.03	0.03	0.02	-	-

NOTES: 20 July 1982 was considered a dry weather sample and 15 September was considered a wet weather sample.

EPA Criteria is from Quality Criteria for Water (1986): also, known as the 'Gold Book'.

Marine Acute Criteria is the highest level of contaminents which would not result in uacceptable effects to sensitive marine aquatic life exposed to the highest one hour average concentration.

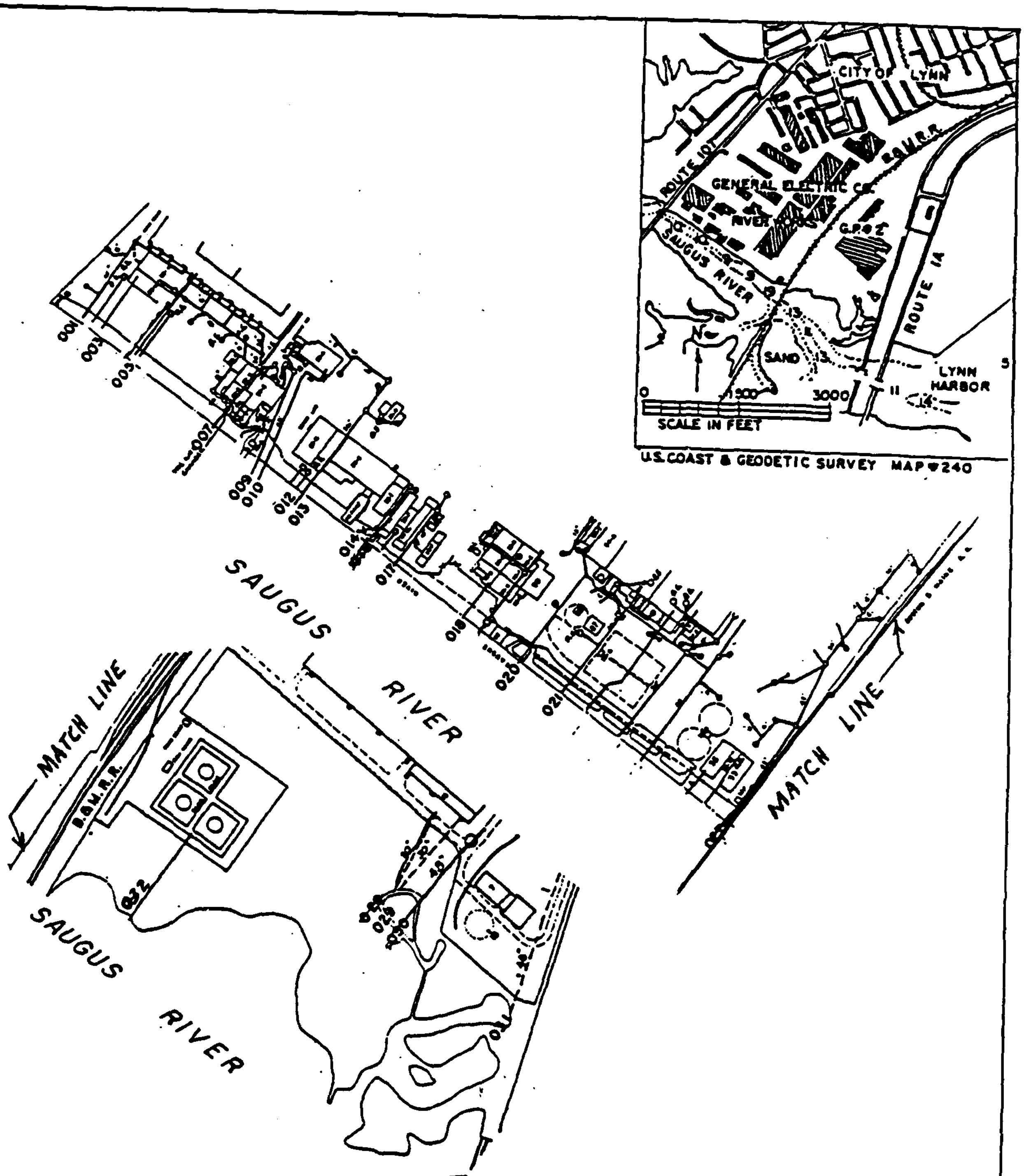
c. Description of Major Pollution Sources

(1) General. There have been several pollution sources identified within the estuary: three are thermal water discharges (figures 11 and 12) - General Electric River Works Plant, RESCO Plant, and Eastern Tool Manufacturing Company; one is an intermittent discharge from Lynn's combined sewer overflow (figure 13); and one is leachate from the Saugus landfill (figure 14).

(2) Thermal Water Sources. According to the MDWPC, there are three major permitted cooling water releases made into the Saugus River. "Major" is defined as having flows greater than 1 million gallons/day (mgd). General Electric has 22 separate discharge locations and the average total permitted rate is approximately 100 mgd; RESCO has one major discharge point with a permitted rate of 60 mgd; Eastern Tool Manufacturing Company has a permitted rate of about 1 mgd.

(a) Eastern Tool Manufacturing Company. Discharges from this source includes noncontact cooling water from industrial processes. Discharge is located upstream of Route 107.

(b) General Electric. Discharges from General Electric generally consist of surface releases of storm water and once-through noncontact cooling water (used for power generation, manufacturing processes, and jet engine testing); however, minor amounts of oils and grease can be discharged. A summary of the effluent limitations from the National Pollutant Discharge Elimination Systems (NPDES) permit are displayed in table 4 and locations of the surface discharge points are displayed in figure 11. The largest discharge volumes listed are from outfalls 14, 18 and 29. Of these three, No. 18's discharge is the most continuous (it is used for cooling power generation equipment); No. 14 operates for an 8-hour period, 3 days a week; No. 29 operates for a 1-hour period, averaging about once a month (both 14 and 29 discharge cooling water which is used for testing manufactured products). General Electric prepared an analysis, "Thermal Discharge from General Electric River Works Power Plant," in September 1973. Temperature data was collected to identify the extent of the heated water dispersion from outfall 18 since this was the only discharge which resulted in a continuous detectable thermal plume. According to the report, the surface discharge was monitored over various tide cycles in August and September 1973. Releases varied from 19 to 32 mgd with a maximum temperature rise of 13° Fahrenheit.



NOTE:

REFER TO TABLE 4 FOR DESCRIPTION OF SOURCES.

LOCATION OF GENERAL ELECTRIC'S NPDES

DISCHARGE LOCATIONS

JUNE 1988

FIGURE 11

FIGURE 12
MAJOR THERMAL DISCHARGES
SAUGUS RIVER ESTUARY
HYD. & WQ. SECT. JUNE 1988

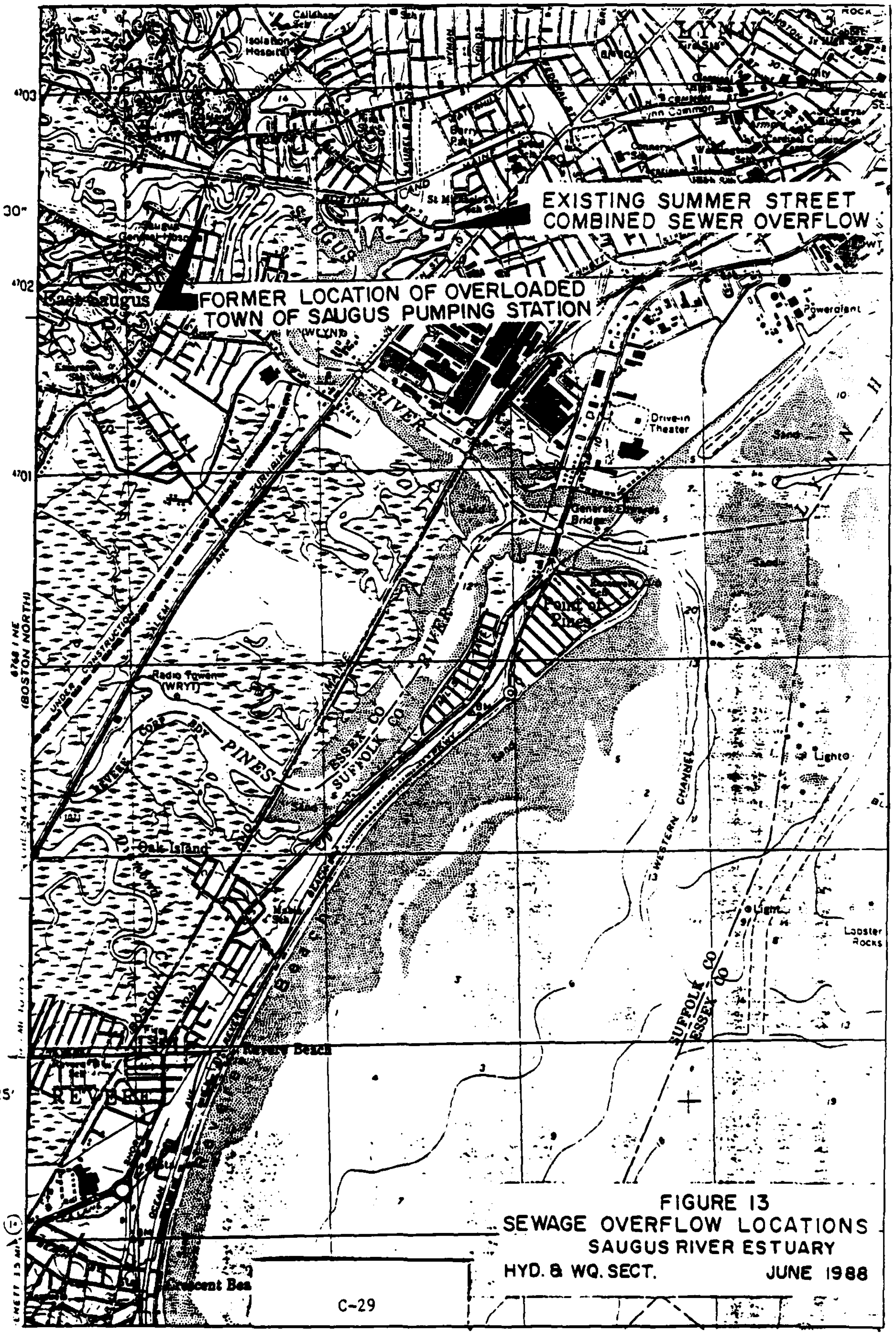


FIGURE 13
SEWAGE OVERFLOW LOCATIONS
SAUGUS RIVER ESTUARY
HYD. & WQ. SECT. JUNE 1988

FIGURE 14
LOCATION OF LANDFILL
SAUGUS RIVER ESTUARY
HYD. & WQ. SECT. JUNE 1988

TABLE 4

SUMMARY OF EFFLUENT LIMITATIONS

GENERAL ELECTRIC CO. RIVERWORKS PLANT
LYNN, MA
(NPDES PERMIT NO. MA 0003905)

OUTFALL NUMBER	DISCHARGE LIMITATIONS **					PRIMARY DISCHARGE SOURCES
	FLOW (MGD)		TEMPERATURE (DEGREES F.)	OIL AND GREASE		
	AVERAGE MONTHLY	MAXIMUM DAILY	MAXIMUM DAILY	AVERAGE MONTHLY	MAXIMUM DAILY	
001	0.01			10	15	ROOF/YARD DRAINS
003	0.80	1.2	74			TEST EQUIPMENT
005	0.02		74			TEST EQUIPMENT
007		1.12	77	10	15	PROCESS COOLING WATER
009		0.071	80	10	15	TEST EQUIPMENT
010	3.60	7.2	90			HEAT EXCHANGERS
012		0.01		10	15	BASEMENT DRAINAGE
013	0.35	0.5	74	10	15	HEAT EX./TEST EQUIP.
014		25.0	90*			TEST EQUIPMENT
015	0.025		90*	10	15	HEAT EXCHANGERS
017	0.005	0.05	84	10	15	YARD DRAINS
018	25.0	33.5	90*			POWER GENERATION
020		16.9		10	15	UNUSED COOLING WATER
021	3.6	7.2	90*	10	15	HEAT EX./TEST EQUIP.
027	1.0	1.5	72	10	15	PROCESS COOLING WATER
028	0.075		75	10	15	YARD DRAINS
029	21.6	43.2	90*			TEST EQUIPMENT
030	0.05		74	10	15	COOLING TOWER
031	0.89	1.6	80	10	15	TEST EQUIPMENT
032				10	15	STORM DRAINS

NOTES: * THE TEMPERATURE OF THE DISCHARGE SHALL AT NO TIME EXCEED A 20 DEGREE F. RISE OVER TEMPERATURE OF THE INTAKE ,BUT ALSO AT NO TIME EXCEED A MAXIMUM OF 90 DEGREE F. AT THE POINT OF DISCHARGE.

** OTHER REQUIREMENTS INCLUDE THAT pH SHOULD BE BETWEEN 6.5 TO 8.5 SU AT ALL TIMES AND THERE SHALL BE NO DISCHARGE OF FLOATING SOLIDS OR VISIBLE FOAM IN OTHER THAN TRACE AMOUNTS.

The plume created did not reach a depth greater than 8 feet nor extend a distance greater than 160 feet from shore. The lateral extent was more variable sometimes reaching greater than 240 feet upstream or downstream depending on the direction of tide.

General Electric recently (September 1985) ended its contract for buying steam from RESCO; however, the amount of cooling water discharged by General Electric has remained the same since General Electric began generating the steam needed from its own onsite boilers.

(c) RESCO. As a result of General Electric's termination of its steam purchasing contract in September 1985, RESCO has opted to generate onsite power using the steam once sold to General Electric. The NPDES permitted noncontact cooling water effluent from RESCO and it is now being discharged from a diffuser placed in the Saugus River about 1,600 feet downstream from the Route 107 bridge and about 500 feet downstream from General Electric's outlet 18 (figure 12). RESCO contracted with Wehran Engineering to define the impacts of the discharge on the river, as part of the requirement for obtaining a discharge permit. Their analysis shows that they were able to meet the following recommended EPA and State guidelines as taken from the NPDES permit:

1. The maximum temperature may not exceed 90° Fahrenheit throughout the receiving water body at the point of discharge.

2. The cross-sectional area enclosed by the 2° Fahrenheit isotherm shall not exceed more than one-third of the cross sectional area of the receiving water body.

3. The maximum temperature rise at the surface should not be more than 4° Fahrenheit.

4. There should be a sufficient volume of water moving past the discharge to significantly dilute the heated effluent. This volume of water may be due to freshwater runoff or tidal flushing or both.

There are fifteen 9-inch diameter ports in the 140-foot long RESCO diffuser. Depth of the water above the ports varies from 9.5 to 14 feet at mean low water and between 6 and 10.5 feet at extreme low water (-3.5 feet below MLW). From Wehran's analysis of thermal dispersion, under the most severe condition (tide level at extreme low water with dry weather riverine flow), the plume at the surface was approximately 50 feet long and 25 feet wide.

(d) Combined Thermal Discharge. RESCO's and General Electric's No. 18 thermal discharge locations are within one-half mile of each other on the Saugus River as shown in figure 12. There has been no study on the cumulative impacts of all the thermal releases including Eastern Tool Manufacturing Company; however, it is noted that General Electric's No. 18, and RESCO's outlet produce a combined total permitted outflow of 100 mgd, (from discussions with General Electric officials, the next two largest discharges, Nos. 14 and 29, are for testing purposes only and it would be extremely rare for all discharges to be operating at once). It is unknown if the upper limit of 60 mgd from RESCO and 40 mgd from No. 18 is ever reached concurrently since sufficient data on coincident flows are not available. RESCO's consultant, Wehran Engineering, addressed this qualitatively by showing that there is enough separation to preclude overlapping plumes but no further analysis was completed. RESCO, through a consultant, is currently investigating the biological impacts of its discharge as a requirement of its permit and EPA and MDWPC are monitoring the results.

Low riverine flow conditions in the Saugus River concurrent with an ebb tide during an extreme neap tide event would produce an estimated average discharge rate of about 3,000 mgd at the mouth of the Saugus River. Conservatively assuming that a combined 100 mgd is being discharged from General Electric's No. 18 and RESCO's outlet, the portion of thermal heated water flow would be about 3.5 percent of the average flow occurring during this extreme low water condition. This estimate is based on USGS quad sheet mapping, coordinated aerial photography/tide stage measurements, and a volume interchange determination using current and tide measurements. For comparison purposes, average annual freshwater river discharge is estimated at 80 cfs or 52 mgd which is a very small component of the total flow.

(3) Combined Sewer Overflow. There is one major combined sewer overflow (CSO) within the Saugus Basin, located off Summer Street in Lynn (see figure 13). As part of a CSO investigation completed by the Lynn Water and Sewer Commission's (LWSC) engineering consultant, the Summer Street overflow was found to drain 27 acres of storm runoff and 546 acres of sewered areas. From estimates developed in the "Lynn Water and Sewer Commission's Combined Sewer Overflow Facilities, Interim Report on Existing Conditions," dated January 1988, the Summer Street overflow is reported to

discharge the most frequently of all the combined sewer overflows in Lynn (40 to 50 times/year). Annual loadings estimated for the Lynn CSO include: Flow - 160 million gallons/year, BOD - 95,000 pounds (lb)/year, Total Solids - 467,000 lb/year, Total Kjeldahl Nitrogen - 19,000 lb/year, Ammonia Nitrogen - 6,000 lb/year, and Phosphorous - 4,000 lb/year.

The recommended plan in their current study is to separate the combined sewers eliminating the combined sewer overflow from the Saugus River.

The old Saugus sewage pumping station located on the banks of the Saugus River near Lincoln Avenue had been a source of sewage overflow during wet weather when leaking sewer pipes caused hydraulic overloading of the station (see figure 13). This problem was corrected when the pumping station was replaced in 1986.

(4) Landfill. A potential major nonpoint source within the estuary is the landfill area located in the salt marsh near the junction of the Pines and Saugus Rivers. The landfill area is comprised of four major sites: the former Daggett and DeMatteo (Saugus) landfill, which occupies almost 200 acres, the RESCO facility which covers approximately 100 acres, the RESCO ash landfill which covers approximately 11 acres, and the GE landfill which covers approximately 10 acres. The entire landfill area is bounded by the Boston and Maine Rapid Transit Line on the east, State Route 107 on the west, and the marsh areas of the Saugus and Pines Rivers on the north and south. Figure 14 shows the approximate location of the landfill area.

Numerous investigations of the landfill have been undertaken; however the majority studied only the Saugus landfill and took place before the RESCO plant was placed in operation in 1975. The most comprehensive document was the Environmental Impact Report on the Saugus landfill project (EIR), prepared for the Massachusetts Department of Environmental Quality Engineering (DEQE) in 1976. In an appendix prepared by John Teal of the National Marine Fisheries Service at Woods Hole, heavy metals, nutrients and polychlorinated biphenyls (PCB's) were found to leach from the landfill but quickly settled out or were absorbed in the marsh muds within 100 to 400 feet downstream from the landfill. In 1980, Massachusetts Coastal Geologist, William Hall, reiterated the findings of the EIR, stating that unquestionable leaching of the pollutants is still taking place including

PCB's, ammonia, cadmium, copper, chromium, nickel, iron, lead, zinc and mercury. Of these contaminants, zinc, cadmium, chromium, lead and PCB's exceeded safe levels for sensitive aquatic life. Mr. Hall noted that the pollutants entered adjacent salt marshes and tidal creeks where they were quickly absorbed by the sediments and in some cases taken up by vegetation. He also noted that since the pollutants are bound up in the fine material, they are essentially immobilized unless the area is disturbed. In his report, he recommends that the site should be closed to further dumping, the area graded, the water quality monitored for changes, and the tidal channels maintained in a clean and free flowing condition in order to benefit from the flushing action of the ocean.

One of the actions prompting the EIR was that owners of the landfill requested that they be allowed to expand into other areas of the marsh. After DEQE turned down their request, the property owners then limited the project to providing cover for final grading, landscaping of the existing landfill area, and disposal of the ash generated by burning refuse at the site. Beginning in 1975, incinerator ash was placed in a special 11-acre site. Use of the ash to cover the old Saugus landfill site was not initiated until after 1982. At that time five out of the original 11 acres had been filled.

The "Phoenix" project, a research study on alternative methods of disposing 200 tons of ash produced each day, was completed by DEQE between 1977 and 1982. Samples of ash were cored from existing filled areas and analyzed. As discussed in the "Summary Update of Research Projects with Incinerator Bottom Ash Residue" by DEQE, dated February 1982, the ash contains elevated levels of soluble mineral salts, alkalinity and heavy metals. Soluble salts included sodium, iron, calcium, potassium, magnesium, phosphorous, chlorides, nitrates and sulfates.

Of the heavy metals, lead and cadmium were generally found to exceed allowable levels for the EP Toxicity Test; however, it was noted that this test produced conditions that would be more severe than what could be expected to occur at the actual site. Lead is an ubiquitous contaminant generally derived from lead based inks used in newsprint. Cadmium is also present due to its use as a stabilizer and pigment in plastic products. Other metals exhibiting high although not toxic levels include copper, zinc, nickel and chromium. In general, the ash was described as being a relatively benign material since it was so highly buffered (pH ranges from 8 to 10 SU). It was also stated that over the short term, there should be no releases of heavy metal due to its buffered condition. However, the long term impacts caused by weathering of the material and the presence of an acid rain condition were concluded as being not so well known. Recent studies completed by DEQE have shown that incomplete burning of refuse has produced small amounts of the chemical, dioxin. According to DEQE, the small amount found does not appear to be a problem.

RESCO is currently involved in a consent agreement with DEQE. One requirement of that agreement is that an assessment of the environmental impacts of handling, storage and the use of ash as a grading material be conducted. RESCO has hired a consultant who is in the process of developing a scope of work to address all the major concerns.

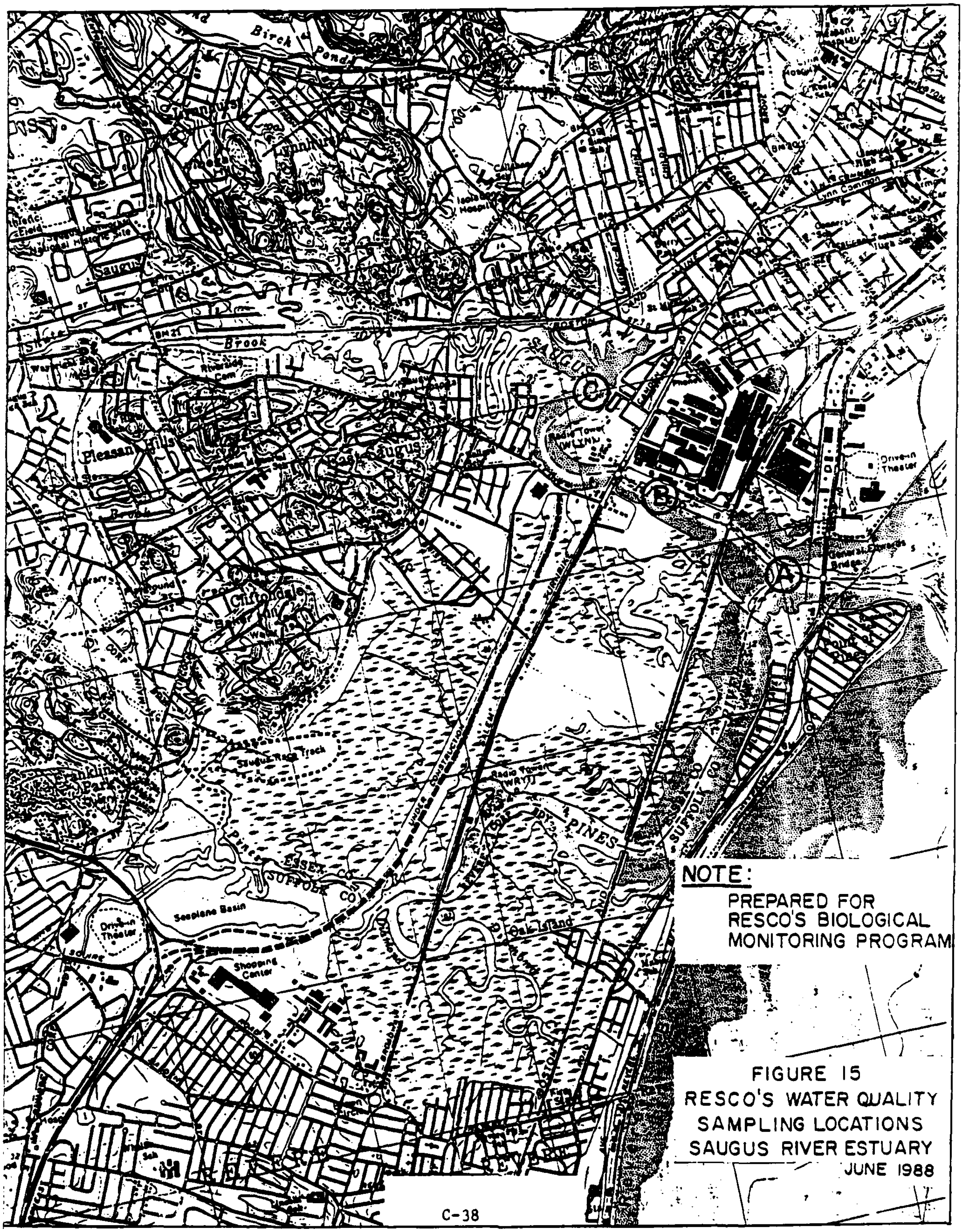
Another small landfill, containing old auto bodies, exists on the Lynn side of the Saugus River just north of Route 107. However, this does not appear to be a significant source of pollutants.

d. Salinity Conditions within the Estuary. There have been many questions regarding the salinity (the amount of salts dissolved in water) of the estuary. In order to understand the existing conditions within the estuary, it is necessary to be familiar with the hydrology. As a result of the sluggish flow from the large upstream wetlands and ponding areas, freshwater inflow into the tidally influenced portion of the Saugus and Pines Rivers is a small component of the estuarine water body. Total drainage area of the Saugus River is small (47 square miles); the freshwater portion of the rivers (located above the MDC Town Line Brook tide gate structure on the Pines River, and above the Saugus Ironworks Park on the Saugus River) amount to approximately 37 square miles. There is no flow gage on either the Saugus or Pines Rivers. Flow estimates were developed based on regional analysis using a nearby gage, on the Parker River at Byfield, Masssachusetts, having similar drainage characteristics. Average annual freshwater flow is estimated at 80 cfs while the flood of record, which occurred in January 1979, produced an estimated freshwater flow rate of 2,000 cfs. By comparison, the estimated total peak outflow measured on 16 April 1987 at the General Edwards bridge for a typical spring tidal condition (tide range +5.6 to -4.7 feet NGVD) was 18,500 cfs or almost 10 times the freshwater flood of record. (The average tidal outflow over the 6-hour ebb tide cycle was approximately 12,000 cfs).

There has not been a great deal of data collected in the past, by others, defining the salinity regime within the estuary. Most salinity data gathered were grab samples taken at a single depth in the area between the mouth of the Saugus River and its confluence with the Pines River. These results showed salinities in the range of 26 to 31 parts per thousand (ppt). More recent data, collected in 1984, as part of RESCO's biological survey program to describe the impacts of its new thermal water diffuser provided information on how salinity varied with depth along the lower part of the Saugus River. At three stations whose locations are shown in figure 15, RESCO's consultant measured salinities as well as temperatures on various sampling dates at the top, middle and bottom of the water column; results are shown in tables 5 and 6. Stations A and B located downstream from Route 107 have salinities in the 27 to 33 ppt range and show complete mixing with surface to bottom variations of less than 1 ppt. Station C located above Route 107 shows only slight density stratification with surface to bottom variations amounting to less than 5 ppt. The temperature data shows similar characteristics.

The Corps collected salinity and temperature measurements at various depths on 20 August 1986 as part of this study at ten different locations as shown in figure 16. For six of the locations (1 through 6), data was gathered generally at one-half to one hour increments. At the other four locations, grab samples were taken at peak low and high tide conditions, when possible. At 1 p.m., peak high tide levels reached approximately 6.0 feet NGVD, which is slightly greater than a mean spring high tide condition. Freshwater inflow was estimated at approximately 50 cfs. Salinity and temperature measurements are presented in tables 7 and 8. Complete water column mixing is present at stations 2, 3, 4 and 5 with nearly complete mixing at stations 1 and 6. At the other 4 stations water depth was generally less than 4 to 5 feet at high tide indicating that these areas are at the upper end of the salinity wedge.

Other salinity measurements were performed during 1 and 2 December 1986 around the time of high tide to try to pinpoint the upper end of salinity wedge intrusion in the upper Saugus River estuary and the Shute Brook areas, (the upper limit of salinity for the Pines River is located at the MDC tide gates at Town Line Brook). Approximate high tide measurements for the two days were 6.5 and 6.8 feet NGVD, respectively. Field crews followed movement of the upper end of the salinity wedge of the approaching high tide where water depths were



NOTE:
PREPARED FOR
RESCO'S BIOLOGICAL
MONITORING PROGRAM

FIGURE 15
RESCO'S WATER QUALITY
SAMPLING LOCATIONS
SAUGUS RIVER ESTUARY
JUNE 1988

TABLE 5

SAUGUS RIVER SALINITY DATA
RESCO BIOLOGICAL MONITORING
PROGRAM - 1984

SALINITY (0/00)

Date (1984)	Station A			Station B			Station C		
	Surf	Mid	Bot	Surf	Mid	Bot	Surf	Mid	Bot
2 Feb	32.4	32.6	32.6	32.3	32.3	32.6		ICED OVER	
2 Mar	30.7	31.3	31.7	25.9	30.5	30.8	13.9	29.3	29.5
16 Mar	31.1	31.6	31.7	30.6	30.6	30.7	19.1	25.7	27.9
2 Apr	29.4	30.5	30.9	28.8	30.1	30.2	23.0	29.0	29.0
16 Apr	29.2	29.2	29.2	28.8	28.7	28.8	26.0	27.4	27.5
30 Apr	27.4	27.8	28.1	27.2	27.4	27.4	23.1	25.5	26.0
14 May	29.4	29.8	29.8	29.2	29.3	29.4	27.0	27.7	28.0
30 May*									
12 Jun	28.4	28.9	29.0	28.5	28.5	28.6	24.6	27.4	27.5
27 Jun	30.0	30.2	30.2	29.4	29.6	29.8	26.3	27.7	28.6
11 Jul	30.6	30.9	31.1	30.5	30.5	30.6	24.9	29.4	29.8
26 Jul	31.3	31.5	31.5	31.0	31.2	31.2	27.8	30.5	30.5
9 Aug	30.6	30.7	30.7	30.5	30.5	30.6	28.7	29.6	29.9
27 Aug	30.8	30.9	31.0	30.5	30.5	30.6	29.5	29.8	29.7
11 Sep	30.9	30.9	30.9	30.1	30.3	30.4	29.4	29.5	29.5
24 Sep	31.8	31.9	31.9	31.6	31.8	31.8	31.2	31.3	31.4
9 Oct	31.7	31.7	31.8	31.3	31.3	31.3	28.4	29.8	30.1
23 Oct	32.0	32.1	32.3	31.7	31.7	31.7	27.3	30.7	30.9
2 Nov	32.3	32.4	32.4	31.7	32.1	32.1	27.1	30.2	30.7
21 Nov	33.0	33.0	33.0	33.0	33.1	33.0	31.9	32.5	32.8
21 Dec	31.6	31.7	31.6	31.6	31.7	31.7	31.3	31.3	31.4

Note: * Conductivity circuit malfunctioned.

TABLE 6

SAUGUS RIVER WATER TEMPERATURE DATA
RESCO BIOLOGICAL MONITORING
PROGRAM - 1984

TEMPERATURE (°C)

Date (1984)	Station A			Station B			Station C		
	Surf	Mid	Bot	Surf	Mid	Bot	Surf	Mid	Bot
2 Feb	-1.3	-1.2	-1.2	-1.3	-1.2	-1.2		ICED OVER	
2 Mar	0.2	0.2	0.0	0.2	0.1	0.0	0.1	0.0	0.0
16 Mar	0.9	0.9	0.8	0.8	0.9	0.7	1.0	0.7	0.8
2 Apr	5.1	4.2	3.8	4.8	4.3	4.1	4.8	4.1	3.8
16 Apr	4.3	4.3	4.3	5.3	4.5	4.5	4.9	4.7	4.8
30 Apr	9.4	9.0	8.8	9.6	9.2	9.3	11.6	10.7	10.2
14 May	7.9	7.3	7.2	9.4	8.1	8.0	10.6	10.1	9.7
30 May	11.0	10.6	10.5	13.0	11.6	11.8	16.0	13.5	13.0
12 Jun	16.5	14.5	14.3	17.0	15.7	15.7	21.3	7.0	16.7
27 Jun	15.8	15.0	14.7	17.0	16.0	15.5	18.3	17.9	17.1
11 Jul	14.2	13.3	12.7	15.1	14.1	13.8	18.8	16.5	15.4
26 Jul	13.2	11.7	11.5	13.7	13.3	13.1	19.8	16.4	16.0
9 Aug	18.7	18.2	17.7	19.3	19.0	18.6	22.9	21.8	21.6
27 Aug	19.3	19.1	19.1	19.7	19.6	19.6	20.9	20.7	20.7
11 Sep	18.0	18.0	18.0	21.1	19.1	18.5	20.1	20.0	20.0
24 Sep	15.0	14.8	14.5	17.2	15.7	15.6	16.9	17.0	16.8
9 Oct	11.8	11.7	11.7	12.0	11.8	11.7	12.0	11.9	11.8
23 Oct	13.1	12.9	12.7	14.6	13.5	13.4	15.2	14.6	14.4
7 Nov	9.5	9.5	9.4	9.7	9.4	9.5	9.2	9.2	9.2
21 Nov	2.9	3.1	3.1	3.4	3.1	3.1	2.7	2.4	2.7
21 Dec	4.1	4.1	4.1	4.6	4.1	4.1	3.8	3.9	3.9

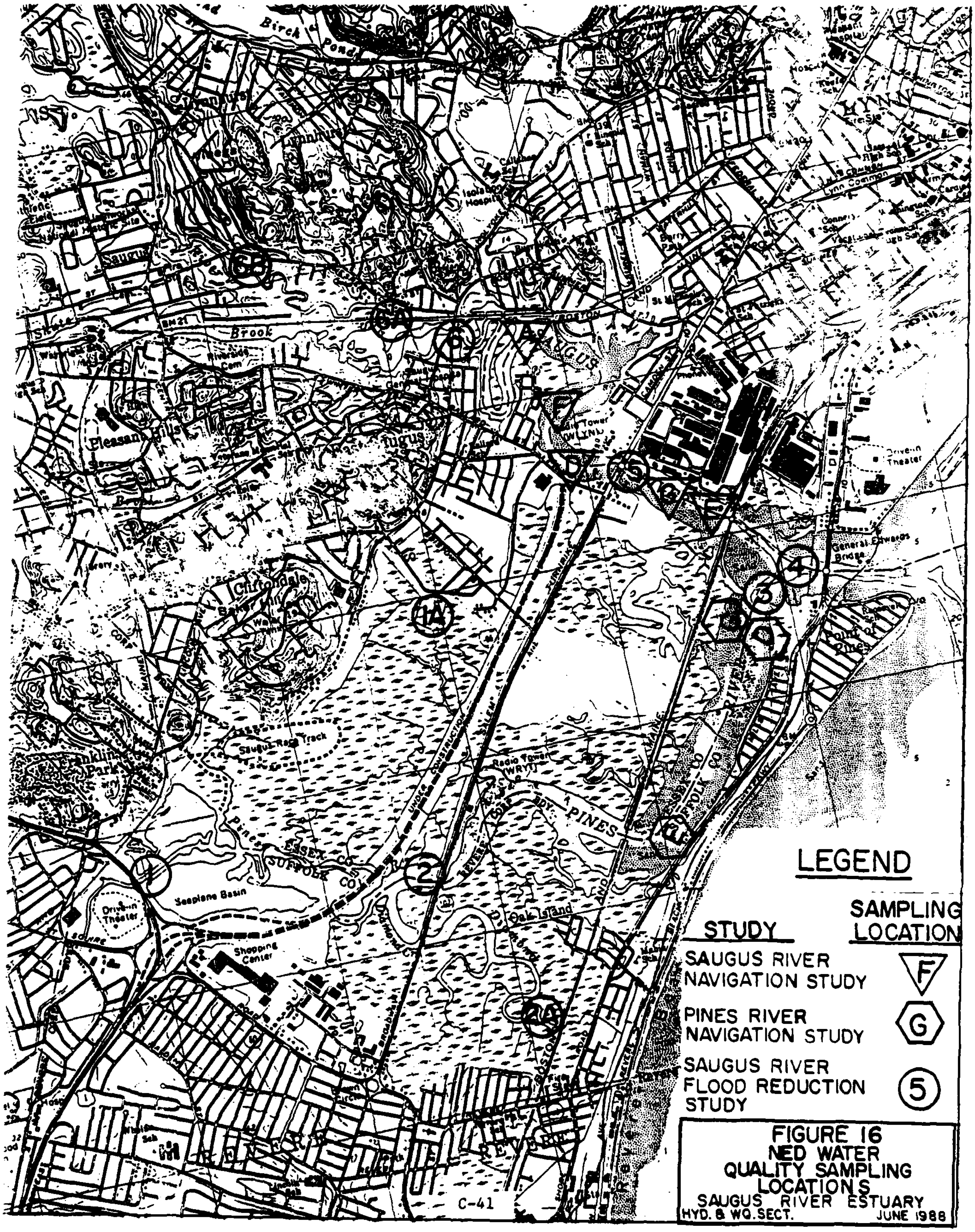


FIGURE 16
NED WATER QUALITY SAMPLING LOCATIONS
SAUGUS RIVER ESTUARY
HYD. & WQ. SECT. JUNE 1988

TABLE 7
SAUGUS RIVER ESTUARY

NED SALINITY DATA
(parts per thousand)

STATION	1			2			3			4			5			6			1A	2A	6A	6B
LOCATION IN THE WATER COLUMN	TOP	MID.	BOT.	TOP	MID.	BOT.	TOP	MID.	BOT.	TOP	MID.	BOT.	TOP	MID.	BOT.	TOP	MID.	BOT.	MID.	MID.	MID.	MID.
TIME																						
6 PM		24.4		28.7	28.8	29.2							25.1		25.1		8.1					
6:30																						
7:00		22.5		28.4	28.4	28.4	28.4		28.7	29.0	28.8	29.0	25.1		25.1		6.2					1.5
7:30							30.4		30.6	29.8	29.8	29.8									3.3	
8:00		19.9		28.8	29.2	28.8	28.7		28.8	29.3	29.4	29.0	26.6		26.6		5.8					
8:30							30.8		30.8	30.1	30.1	29.9				7.9		8.6				
9:00		18.1		28.8	28.7	28.8	29.6	29.6	29.2	29.7	29.7	29.7		28.8								
9:30							31.0	31.0	31.0													
10:00		25.9		29.6	29.6	28.8	29.9	29.8	30.3	29.9	30.1	29.9	28.1	28.4	28.7	17.7		18.8				
10:30							31.3	31.3	31.3	30.2	30.2	30.2	29.6	29.4								
11:00	28.1		28.1	29.6	29.7	29.9	30.3	30.3	30.3	30.3	30.3	30.2	29.6	29.6	29.6	25.5		25.9				
11:30							31.3	31.3	31.3	30.4	30.4	30.4							3.8			
12 PM	28.2		28.4				29.9	29.9	30.3	30.4	31.0	30.4	29.6	29.6	29.6	22.4	26.7	27.0		25.2		7.5
12:30							31.4	31.4	31.4													
1:00	28.4		29.2				29.9	29.9	29.9	30.2	30.2	30.3	29.6	29.6	29.6						27.4	
1:30				29.9	29.9	29.9				30.7	30.6	30.7										
2:00	28.4		29.2				29.7		29.9	30.3	30.4	30.4	29.6	29.6	29.6	21.3		26.2				
2:30				29.9	29.9	29.7	31.8		31.9	31.0	31.0	31.0										
3:00	28.4		29.6	29.6	29.6	29.7	29.9		29.9	30.1	30.3	30.4	29.5	29.6	29.5	25.0		24.7				
3:30							31.8		32.0													
4:00	28.4		29.2	29.5	29.2	29.5	29.9		29.6	30.2	30.1	29.6	28.8	29.2	29.2		21.0					
4:30							32.1		32.2	30.9	31.0	31.1										
5:00		28.4		29.2	29.2	29.2	29.5		29.6				27.5		27.0		13.9					
5:30							32.1		32.2	29.0	29.6	30.3										
6:00							32.1		32.1	30.6	31.0	31.0	25.5		26.2							

NOTES 1 THE FOLLOWING ARE THE APPROXIMATE TIMES FOR HIGH AND LOW TIDES

STATIONS	LOW	HIGH	LOW
3, 4, 5, 6, 6A, 6B	6:00 AM	12:30 PM	6:30 PM
1, 2, 1A, 2A	6:30 AM	1:00 PM	7:00 PM

2. HIGH TIDE LEVELS REACHED 5.8' NGVD AT STATION 1 AND 6.0' NGVD AT STATION 5.

3. SAMPLES COLLECTED ON AUGUST 20, 1986.

TABLE 8
SAUGUS RIVER ESTUARY
NED WATER TEMPERATURE DATA
(DEGREES FAHRENHEIT)

STATION	1			2			3			4			5			6			1A	2A	6A	6B
LOCATION IN THE WATER COLUMN	TOP	MID.	BOT.	TOP	MID.	BOT.	TOP	MID.	BOT.	TOP	MID.	BOT.	TOP	MID.	BOT.	TOP	MID.	BOT.	MID.	MID.	MID.	MID.
TIME																						
6 AM		54.0			62.0									61.0			66.0					
6:30																						
7:00		55.0			64.0		66.0		66.0				64.0		64.0		68.0					68.0
7:30				64.0	66.0	64.0	66.0		66.0	67.0	67.0	67.0									70.0	
8:00		55.0		60.0	66.0	66.0	66.0		66.0	67.0	67.0	67.0	66.0		66.0		70.0					
8:30							67.0		66.0	67.0	67.0	67.0										
9:00		57.0		68.0	66.0	66.0	65.0	65.0	65.0	65.0	65.0	65.0		63.0		70.0		72.0				
9:30							64.0	64.0	64.0													
10:00		57.0					64.0	64.0	64.0	66.0	65.0	65.0	63.0	63.0	64.0	70.0		70.0				
10:30				66.0	66.0	66.0	64.0	64.0	64.0	65.0	65.0	65.0										
11:00	55.0		55.0	66.0	66.0	66.0	64.0	64.0	64.0	65.0	65.0	65.0	63.0	64.0		71.0		71.0				
11:30							64.0	64.0	64.0	65.0	65.0	65.0							72.0			55.0
12 PM	57.0		57.0				65.0	64.0	64.0	65.0	65.0	65.0	63.0	63.0	63.0	73.0	73.0	72.0		73.0		
12:30							65.0	65.0	65.0													
1:00	55.0		55.0				65.0	65.0	65.0	66.0	66.0	65.0	68.0	64.0	64.0						75.0	
1:30				66.0	66.0	66.0				66.0	65.0	65.0										
2:00	55.0		55.0	64.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	66.0	64.0	66.0	64.0	74.0		74.0				
2:30							66.0		66.0	68.0	67.0	67.0										
3:00	59.0		59.0	68.0	68.0	66.0	66.0		66.0	67.0	67.0	67.0	64.0	64.0	64.0	75.0		75.0				
3:30							67.0		67.0													
4:00	59.0		59.0	68.0	66.0	66.0	67.0		67.0	70.0	69.0	68.0	68.0	68.0	68.0		74.0					
4:30							68.0		68.0													
5:00		57.0		68.0	66.0	66.0	70.0		69.0	71.0	70.0	69.0	68.0		66.0		72.0					
5:30							70.0		70.0	71.0	70.0	70.0										
6:00							70.0		70.0	71.0	70.0	70.0										

NOTES 1 THE FOLLOWING ARE THE APPROXIMATE TIMES FOR HIGH AND LOW TIDES

STATIONS	LOW	HIGH	LOW
3, 4, 5, 6, 6A, 6B	6:00 AM	12:30 PM	6:30 PM
1, 2, 1A, 2A	6:30 AM	1:00 PM	7:00 PM

2. HIGH TIDE LEVELS REACHED 5.8' NGVD AT STATION 1 AND 6.0' NGVD AT STATION 5.

3. SAMPLES COLLECTED ON AUGUST 20, 1986.

generally less than 6 feet. The criteria used for identifying the upper edge of the salt wedge was to use salinity values less than 0.5 ppt as a cutoff point. It was determined that for these particular tide levels, salinity did not extend any further than just upstream from the southerly Saugus Ironworks historical site boundary line on the Saugus River and about 500 feet south of the Central Street bridge on Shute Brook (see plate 1). The streambeds for both these water bodies rise very quickly soon after those points and even extreme storm tides (10 feet NGVD) would only extend upstream from this point less than 1,000 feet. It was also noted that there did not appear to be any salinity resistant-vegetation existing above the Saugus River historical site or above the Central Street bridge. On these days, salinity measurements in the Saugus River at the Lincoln Avenue bridge showed nearly complete mixing during peak high tide conditions while the water column in the Saugus River at the Hamilton Street bridge and in Shute Brook about 1,000 feet above the Lincoln Avenue bridge showed definite stratification in the relatively shallow waters (less than 6 feet deep).

Other observations made as a result of the Corps salinity measurement program during low freshwater flow conditions include: (1) the lower Pines and Saugus Rivers (east of Route 107) are completely mixed and the freshwater contribution is so minor it acts as a homogeneous saline water body, (2) due to its enormous surface area and its small freshwater drainage contribution (total drainage area of the Pines River = 8.9 square miles), the Pines River in the vicinity of the Seaplane Basin also acts as a homogeneous saline water body, (3) the narrow bridges on both the Saugus and Pines Rivers cause increased turbulence and result in complete mixing at the structures, and (4) the only salinity stratification that occurs during dry weather conditions is in those areas upstream from the Lincoln Avenue bridge in the Saugus River and Shute Brook.

The movement of the salt wedge for Shute Brook and the upper Saugus River is dependent upon the freshwater flow rate and the tidal range. Assuming a storm hydrograph with a 12-hour base and a peak flow rate of about 2,000 cfs (the estimated peak flow rate of record) the freshwater volume produced would be about 1,000 acre-feet which is equivalent to the volume in the upper end of the Saugus River tidal estuary above Fox Hill bridge up to a height of 6.5 feet NGVD.

e. Corps Water Quality Data

(1) General. For the purposes of defining impacts for this study and for NED navigation studies of the Saugus and Pines River, the Corps Water Quality Lab has collected water quality data over the past several years. Water column grab samples were collected and metals, nutrients and hydrocarbons analyzed during various times in the 1982-1984 period as part of the navigation investigations. Water column grab samples were also collected at various times over a full tide cycle on 20 August 1986 under dry weather conditions for this study and metals, DO, pH, coliform, nutrients, suspended solids, oil and grease, color and turbidity parameters were analyzed. Figure 16 shows locations of the samples gathered by the Corps and table 9 summarizes results of the water quality sampling programs for the Corps' Saugus and Pines Rivers navigation studies. Tables 10 and 11 present results of the water quality analyses for the August 1986 program conducted as part of this study. Table 12 presents EPA's water quality criteria for toxic substances in a saltwater environment ("Gold Book Criteria").

Review of the data collected shows agreement with the conclusions reached by the MDWPC in their 1982 Water Quality Management Report for the basin, i.e., there does not appear to be any major pollutant sources attributable to industrial activities; in general, despite dense urbanization of the basin, BOD, DO, nutrients, and suspended solid levels are indicative of good water quality during dry weather conditions; the upper portion of the tidal estuary which consists of the tidal area above Lincoln Avenue on the Saugus River and above the I-95 embankment on the Pines River, is the most severely impacted by sources of domestic sanitary pollution; and this sanitary pollution is carried over to the lower part of the basin where as a result of tidal flushing action, high pollutant concentrations are diminished. Other problems noted during the data collection included high coliform levels in the upper tidal estuary during dry weather, low DO levels in the upper Pines River and occasional high concentrations of certain heavy metals throughout the tidal estuary. Further discussion of data is presented below.

(2) Coliform. During the August 1986 sampling total coliform counts varied from 30 to over 11,000 colonies/100 ml and fecal coliform levels ranged from 2 to 11,000 colonies/ 100 ml. The highest levels, which exceeded state criteria, generally occurred at the upper ends of the tidally influenced portions of the water body during a low tide condition (note results for sampling stations 1, 2a, 6, 6a and 6b in table 10). High coliform levels are also

TABLE 9
WATER QUALITY SAMPLING RESULTS
NED'S NAVIGATION INVESTIGATIONS

STATION	NAVIGATION STUDY	WATER QUALITY PARAMETERS															
		NITRATE/ NITRITE (MG/L)	AMMONIA (MG/L)	TOTAL PHOSPH. (MG/L)	PCB (PPB)	OIL AND GREASE (MG/L)	METALS (IN PPB)										DATE OF SAMPLE
							ARSENIC	CADMIUM	CHROMIUM	COPPER	LEAD	MANGAN.	MERCURY	NICKEL	ZINC		
A	SAUGUS RIVER		0.61	0.13		<1.0	<3.3	<0.5	<0.5	<1	1	20	1.3	10	27		9/82
D	SAUGUS RIVER		0.48	0.11	<0.1	<1.0	<3.3	<0.5	<0.5	4	<1	40	1.7	<10	65		9/82
E	SAUGUS RIVER		0.38	0.09	<0.1	<1.0	<3.3	<0.5	1.0	<1	2	10	1.9	<10	<25		9/82
F	SAUGUS RIVER	0.72		0.10	2.9	0.5	<5.8	<0.38	<6.2	9.6	<2.5	-	<0.5	<8.2	23		4/84
G	SAUGUS RIVER	0.71		0.04	<0.2	225	6.0	<0.38	<6.2	<8.3	<2.5	-	<0.5	<8.2	32		4/84
D	PINES RIVER	0.19			<2	<1.0	<1.5	<0.2	<2	21	2	4	5.9	4	78		12/82
E	PINES RIVER	0.25				<1.0	1.5	1	2	15	10	7	1.8	6	102		12/82
G	PINES RIVER	0.18		0.04		<0.43	6.9	<0.38	<6.2	<8.3	<2.5	-	<0.5	14	25		4/84

TABLE 10

SAUGUS RIVER ESTUARY
NED WATER QUALITY DATA

STATION	TIDE	TOTAL COLIFORM (NO./ 100 ML)	FECAL COLIFORM (NO./ 100 ML)	DISS. OXYGEN (MG/L)	BOD (MG/L)	pH (SU)	NUTRIENTS: TOTAL PHOSPH. (MG/L)	NUTRIENTS: NITRATE/ NITRITE (MG/L)	NUTRIENTS: AMMONIA (MG/L)	TURBID. (JTU)	APPAR. COLOR (PT.-CO. UNITS)	OIL AND GREASE (MG/L)	PCB (PPB)	METALS (in PPB): ARSENIC	CADMIUM	CHROMIUM	IRON	LEAD	MERCURY	ZINC
1	LOW	2460	420	3.3		6.3	0.11	0.22	0.39	3.9	20	0.32		<2	<0.5	2.3	170	<1.5	1	55
	MID						0.11	0.45	0.56					<2	<0.5	2.8	0.4	<1.5	1	35
	HIGH	290	36	4.2		6.5	0.07	0.08	0.20	2.9	10	0.07		<2	<0.5	1.8	<0.1	<1.5	2	15
	MID						0.06	0.15	0.19					2	<0.5	3.8	<0.1	<1.5	<1	62
	LOW						0.03	0.12	0.24	2.1	15	0.24		2	<0.5	3.8	<0.1	<1.5	1	70
2	LOW	1480	160	3.4		7.2	0.02	0.10	0.19	4.0	15	0.15		2	<0.5	4.5	0.14	<1.5	1	25
	MID						0.09	0.11	0.20					<2	<0.5	4.7	<0.1	<1.5	1	17
	HIGH	65	4	4.8	0.0	7.6	<0.1	0.14	0.10	3.9	10	0.09		<2	<0.5	5.7	<0.1	<1.5	1	<2
	MID						0.06	0.06	0.10					<2	<0.5	3.5	<0.1	<1.5	1	3
	LOW						0.08	0.13	0.19	2.8	15	0.14		<2	<0.5	6.2	<0.1	<1.5	1	17
3	LOW	560	56	7.1		7.4	0.06	0.08	0.16	3.6	10	0.09	<0.03	2	<0.5	2.5	<0.1	<1.5	<1	31
	MID			9.4		7.9	0.02	0.20	0.17					2	<0.5	3.2	<0.1	<1.5	1	88
	HIGH			8.5	0.9	7.9	0.05	0.05	0.06	1.0	5	0.14	<0.03	<2	<0.5	3.1	<0.1	<1.5	1	5
	MID			8.8		7.7	0.04	0.03	0.04					<2	<0.5	1.8	<0.1	<1.5	1	<2
	LOW			7.1		7.6	0.06	0.08	0.16	3.3	10	0.14		2	<0.5	3.6	<0.1	<1.5	1	4
4	LOW	1440	70	4.0		7.3	0.10	0.43	0.23	1.6	10	0.17	<0.03	<2	<0.5	2.6	<0.1	<1.5	1	81
	MID			7.6		7.4	0.08	0.09	0.15					<2	<0.5	2.3	0.11	<1.5	2	38
	HIGH	29	2	7.0	0.6	7.7	0.04	0.05	0.14	0.8	5	0.10		<2	<0.5	<1	<0.1	<1.5	1	<2
	MID			7.0		7.6	0.03	0.45	0.07					<2	<0.5	4.7	<0.1	<1.5	1	<2
	LOW			6.2		6.6				1.2	5									
5	LOW	760	220	5.4		7.0	0.10	0.29	0.26	3.2	15	0.20	<0.03	2	<0.5	4.0	0.11	<1.5	2	27
	MID			5.3		7.5	0.10	0.67	0.21					3	<0.5	4.5	0.15	<1.5	1	8
	HIGH	170	30		1.2		0.05	0.22	0.18	3.3	5	0.14	<0.03	<2	<0.5	4.3	<0.1	<1.5	<1	22
	MID						0.06	0.12	0.11					<2	<0.5	5.3	<0.1	<1.5	1	2
	LOW						0.09	0.20	0.20			0.31		<2	<0.5	2.7	<0.1	<1.5	2	22
6	LOW	5200	640	6.6		6.2	0.12	0.75	0.21	4.0	40	0.12		<2	<0.5	<1		<1.5	10	24
	MID						0.15	0.73	0.20					<2	<0.5	4.3		<1.5	6	60
	HIGH				1.2					1.2	10									
	MID						0.09	0.24	0.20			0.25		2	<0.5	4.8		<1.5	1	13
	LOW						0.09		0.04	1.6	25			2	<0.5	6.3		<1.5	1	22
1A	HIGH	260	260	8.7		6.0	0.05		0.07					<2	<0.5	10.9	0.29	<1.5	2	57
2A	HIGH	11300		4.1		6.9	0.11	0.13	0.23					2	<0.5	4.1	<0.1	<1.5	1	15
6A	LOW	3600	1100	6.3		6.2													1	
	HIGH	752	87	6.8		6.4														
6B	LOW	944	690	9.7		6.4														
	HIGH	1680	950	7.2		6.4														

NOTE: ALL PARAMETERS WERE MEASURED FROM THE MIDDEPTH OF THE WATER COLUMN ON AUGUST 20, 1986.

TABLE 11

SAUGUS RIVER ESTUARY
TOTAL SUSPENDED SOLIDS
(IN MG/L)

STATION	1			2			3			4			5			6		
LOCATION	TOP	MID.	BOT.	TOP	MID.	BOT.	TOP	MID.	BOT.	TOP	MID.	BOT.	TOP	MID.	BOT.	TOP	MID.	BOT.
TIME																		
6:00 AM		8.4		16.5	11.4	18.8							14.9		15.1		9.0	
6:30																		
7:00		8.9		6.0	12.4	7.5	15.8		14.2	9.8	11.5	11.7	12.8		11.2		14.9	
7:30																		
8:00		9.0		17.0		13.8	15.7		15.9	11.4	27.9	9.9	14.6		34.1		24.7	
8:30																		
9:00		12.5		9.4	7.6	12.5	14.0	14.4	24.0	20.9	11.1	14.5		17.1		17.8		15.1
9:30																		
10:00		13.2		15.2	19.9	12.3	10.9	16.3	12.4	16.0	9.2	25.5	15.0	15.2	17.9			10.5
10:30																		
11:00	9.4		8.6	13.0	16.6	37.0	10.2	12.2	13.7	8.7	7.9	29.3	12.4	15.5		12.6		12.4
11:30																		
12:00 PM	28.6		11.9				16.2	13.0	10.1	21.9	31.4	9.2	17.6	13.4	12.4	8.2	13.2	11.4
12:30																		
1:00	9.7		10.4				9.4	8.2	7.3	10.6	9.8	7.2	9.0	12.8	11.8			
1:30				13.0	10.7	13.4												
2:00	7.1		11.4				8.4		10.4	13.3	9.0	8.6	12.1	12.6	7.2	10.2		11.0
2:30				9.8	9.2	8.6												
3:00	6.7		20.1	11.2	11.3	11.6	20.0		12.0	7.9	24.5	28.7	9.2	10.5	11.4	11.6		10.7
3:30																		
4:00	7.0		25.0	9.1	14.5	30.8	12.5		11.7	7.5	7.6	10.4	8.6		10.5		8.5	
4:30																		
5:00		10.0		10.2	11.8	12.2	13.6		12.9	8.5	11.8	9.6	11.7		8.1		9.6	
5:30																		
6:00													24.0		24.9			

NOTES: 1 THE FOLLOWING ARE THE APPROXIMATE TIMES FOR HIGH AND LOW TIDES

STATIONS	LOW	HIGH	LOW
3, 4, 5, 6, 6A, 6B	6:00 AM	12:30 PM	6:30 PM
1, 2, 1A, 2A	6:30 AM	1:00 PM	7:00 PM

2. HIGH TIDE LEVELS REACHED 5.8' NGVD AT STATION 1 AND 6.0' NGVD AT STATION 5.

3. SAMPLES COLLECTED ON AUGUST 20, 1986.

4. VOLATILE SUSPENDED SOLIDS FOR THESE SAME SAMPLES RANGED FROM 0.6 MG/L TO 5.4 MG/L; AVERAGING ABOUT 3 MG/L.

TABLE 12

ENVIRONMENTAL PROTECTION AGENCY'S
WATER QUALITY CRITERIA (1986)
TOXIC LEVELS FOR SENSITIVE AQUATIC LIFE
("GOLD BOOK CRITERIA")

PARAMETER	MARINE * ACUTE CRITERIA (PPB)	MARINE ** CHRONIC CRITERIA (PPB)
CADMIUM	43	9.3
CHROMIUM (HEX.)	1100	50
COPPER	2.9	2.9
LEAD	140	5.6
MERCURY	2.1	0.025
NICKEL	140	7.1
ZINC	170	58
PCB	10	0.03

NOTES: * Marine Acute Criteria is the highest level of contaminants which would not result in unacceptable effects to sensitive marine aquatic life exposed to the highest one hour average concentration.

** Marine Chronic Criteria is the highest level of contaminants which would not result in unacceptable effects to sensitive marine aquatic life exposed to the highest four day average concentration.

carried downstream in the Saugus and Pines River where state criteria for total coliform were exceeded at stations 2, 4 and 5 during low tide conditions. Since this sampling had occurred during a period when there had been no antecedent rainfall for several days it appears, as suggested by the MDWPC, that direct sewage overflows or defective septic systems are draining into Shute Brook, upper Diamond Creek, Town Line Brook and possibly the Saugus River above the Lincoln Avenue bridge. Coliform levels are diluted significantly during high tides, such that state saltwater criteria were met at all stations with the exception of stations 2a, 6a and 6b.

(3) Dissolved Oxygen and BOD. Dissolved oxygen levels were measured during low and high tide conditions in August 1986. The results generally show that during extreme low tide conditions there is a low DO problem in the Saugus and Pines Rivers above Route 107. During high tide measurements, most areas meet the minimum 6 mg/l state criteria with the exception of the upper Pines River and upper Diamond Creek area. BOD measurements were made on grab samples taken at high tide. All results are less than 1.5 mg/l and show no indication of a large pollutant source. It is unknown at the present time what causes the low dissolved oxygen; however, benthic organism interaction with built-up organic sediments may be a significant problem. Also, algae buildup in the upper portions of the Saugus and Pines may cause DO depletion during summer weather. Nutrient levels are sufficient to support rapid algae growth and, in the case of the upper Pines River above the I-95 embankment, tidal interchange is somewhat limited by the weir-like restriction which currently exists as the opening for the I-95 embankment. Data collected during a wet-weather period by LWSC's consultant indicate that storm events have more of an impact on the DO of the upper non-tidal portions of the basin than on the tidal portion where runoff is diluted by the tidal prism.

(4) pH. Those areas inundated by large amounts of saltwater have high pH as depicted by the results at stations 2 through 5; pH generally above 7, rises as high as 8.0 standard units (SU). Lower pH below the state criteria of 6.5 SU occurs in the upper areas of the Saugus and Pines Rivers and in the small streams that drain into the estuary during times of low tide. Since there are no known industrial discharges in the upper basin, the low pH may either be related to the "acid rain" phenomena which has been documented throughout New England or due to organic acids which are produced in peat areas of the upstream wetlands. The soils of the freshwater portions of the upper drainage basin have insufficient buffering capacity to raise the pH of acid runoff.

(5) Nutrients. Total phosphorous, nitrate/nitrite nitrogen and ammonia were measured during both the navigation studies and in August 1986. Total phosphorous levels ranged from a low of .03 mg/l to a high of 0.15 mg/l. According to the 1986 data the higher concentrations occurred at times of low tide and at the upstream ends of the tidal marsh indicating that the pollution source may be from combined sewer overflows or septic system failures. Nitrate/ nitrite levels ranged from 0.045 to 0.75 mg/l and ammonia levels ranged from 0.04 to 0.61 mg/l; both parameters generally followed the same occurrence pattern as previously described for phosphorous. None of the samples analyzed indicate that there is a major point source of domestic wastewater occurring during dry weather conditions. Further analysis undertaken during the wet-weather sampling program of the LWSC combined sewer overflow study indicates that there are no other known major point sources other than from the CSO.

(6) Suspended Solids and Sediment Movement. Suspended solids were collected throughout the 12-hour tide cycle at top, middle and bottom of the water column for stations 1 through 6 (table 11). Data show low amounts of total solids throughout the tide cycle with values ranging from 7 to 31 mg/l; solids concentrations averaged 13 mg/l and the volatile portion averaged about 3 mg/l. There was not a significant variation from top to bottom at any of the sites indicating that there was only a minor amount of suspended sediment moving along the bottom. The largest difference between the surface to bottom values occurred as expected near station 4 where the bridge constriction caused higher velocities. Still the variations were small ranging from 16 mg/l at the top to 26 mg/l at the bottom for the floodtide and from 8 to 29 mg/l for the ebb tide.

The suspended solids results reinforce what has been suspected, i.e., any downstream migration of sediment from the upper basin will be minimal during low freshwater flow conditions because of hydrologically slow basin characteristics (short stream lengths interconnected by large wetland areas). It is likely that if there is any movement of sediment from the upper basin at all, it will take place during major freshwater runoff events. Since normal freshwater contribution is minimal with respect to the tidal interchange volume, erosion and sedimentation in the saltwater estuary in the lower basin will be controlled by tidal fluctuations. There was very little sediment movement measured in the saltwater wetlands during a normal tide range condition on the 20 August 1986 sampling date (tide range 10.6 feet on flood, 9.8 feet on ebb). It is assumed that if suspended solids concentrations resulting from sediment movement are significant at all, they would be observed during a major tidal storm event. Another conclusion based on current and tidal measurements is that sediment movement in both the Pines and Saugus Rivers upstream from Route 107 will probably not be as significant as that downstream since there are restrictive bridge openings in this area which dampen rapid tidal changes.

Sediment accretion at the mouth of the Saugus River near Point of Pines has also been noted from discussions with long term residents. Sediment seems to be accumulating at the mouth of the Saugus River as a result of the northerly movement of material along Revere Beach (littoral process) rather than from the outward transport from the river. More detailed analysis is presented in the Hydraulics and Hydrology and Evironmental Appendices.

(7) Turbidity and Color. Turbidity and apparent color measurements were made at middepth of the water column for high and low tide conditions for stations 1 through 6 during the August 1986 sampling (table 10). Turbidity values were low, ranging from 0.8 to 4 Jackson Turbidity Units (JTU). Apparent color levels showed significantly more variation, ranging from 5 Standard Units (SU) up to 40 SU. The highest levels were recorded at the most upstream stations (1 and 6), with station 6 having a 40 SU reading at dead low tide, a value which might be expected from a freshwater wetland type environment. Apparent color values in the well mixed estuary averaged around 10 SU.

(8) Oil and Grease. Oil and grease measurements were collected during low and high tide conditions during the August 1986 sampling and grab samples were also taken during the Pines and Saugus Rivers navigation study. In general, the levels of oils and grease are low, less than 1 mg/l with the exception of a high reading (225 mg/l) taken at station G on the Saugus River just downstream from the Route 107 bridge (see table 9). This high value could have occurred as a result of a small spill from one of the commercial boats or marinas located upstream. It is unlikely that this is a continuous source since standard EPA elutriate tests were also completed on sediments taken from throughout the proposed channel for the Saugus and Pines Rivers navigation studies and these results generally show oil and grease concentrations of less than 1 mg/l with 12 mg/l being the most ever measured.

(9) PCB's. Grab samples for PCB's were collected during both the August 1986 sampling and the navigation studies. Analysis shows that PCB concentrations were in general less than 0.03 ppb (parts per billion) except for one sample taken during the navigation study upstream from Route 107 on the Saugus River when a level of 2.9 ppb was measured. This level is higher than EPA's chronic criteria but less than the acute criteria necessary to protect sensitive aquatic life.

(10) Metals. Grab samples were collected at various times over the full tide cycle on 20 August 1986 for stations 1 through 6 and singular grab samples were collected for various stations for the navigation studies. The results show that there were a number of metals exceeding EPA's chronic criteria to protect sensitive marine aquatic life although the less stringent acute criteria were usually met. Mercury appears to exceed the chronic criteria frequently while other metals showing occasional exceedances include copper, zinc, lead and nickel. Acute criteria is also occasionally exceeded by copper. It is unknown what effects, if any, these elevated levels are having on the environment; marine life may have adapted to the higher levels or perhaps the metals are in a form not easily assimilated by the organisms. Mercury levels ranged from less than 1 ppb to as high as 10 ppb, the higher levels occurring at station 6 at the Lincoln Avenue bridge. Copper measurements made during August 1986 were discarded since there appeared to be contamination in the sampling apparatus; however, copper data collected as part of the navigation studies showed levels varying from less than 1 ppb to 21 ppb with the highest levels occurring in the lower Pines River area. Lead displayed only one exceedance of the chronic criteria in the lower Pines River area. The rest of the measurements were generally less than 1.5 ppb. Zinc levels range from less than 2 ppb to 102 ppb with only a few samples exceeding EPA's chronic criteria. The higher levels appear to be more prevalent during low tide and generally closer to the freshwater source. Nickel was measured during the navigation studies and showed values ranging from 4 ppb to 23 ppb, slightly exceeding EPA's chronic criteria a number of times on the lower Saugus River. Cadmium, chromium and arsenic were measured during August 1986 and values were less than EPA's chronic criteria levels.

3. FUTURE WATER QUALITY

a. General. The majority of the text on future water quality conditions will deal with implementation of the Regional Floodgate Plan since it raises the most questions relative to potential changes from existing conditions. The nonstructural scheme should have no impact since construction will be minor and not take place in the wetlands or waterways of the study area.

The Local Protection Plan (LPP) alternative (figure 2) may have minor long term impacts on water quality since the required earthen dikes and/or concrete walls generally would have to be constructed near the wetland/upland boundary. The implementation of the LPP scheme would result in a temporary increase in turbidity during construction and a permanent loss of about 40 acres of nutrient, organic, and metal absorbing wetland area (about half vegetated). There would be no change in nonstorm related flowage areas; therefore, no change should be noticeable in the normal flushing characteristics of the estuary. During storms, velocities in the channel will increase slightly in some areas if the dikes associated with the plan reduce the flowage area. Any increase in turbidity relating to velocity increases should be minor.

The Regional Floodgate Plan components (figure 3), of which the tidal floodgate is the main constituent, will directly impact no vegetated wetland but will, due to the physical constriction of the structure at the river's mouth, produce minor alteration in tidal levels and flushing in the estuary. The degree of change is related to the size of the openings and their locations. Analysis has shown that the proposed navigation and flushing gate openings are of sufficient size and depth so that they will cause only minor changes to the flushing characteristics of the estuary in the open position. Because of the concern expressed about flushing changes and its impact on water quality, significant detail has been presented in this text and in the Hydrology and Hydraulics Appendix to describe the differences. Changes in currents will, in general, be restricted to within a few hundred feet of the gate structure. During relatively infrequent periods of gate closure for ocean storms, tidal flushing of the estuary will cease for a few hours. During this period freshwater runoff will be retained in the estuary. Normal flushing will rapidly resume after several hours of gate closure when storm tides recede below damaging levels.

b. Current and Flushing Impacts of the Tidal Floodgate

(1) General. A hydrologic and hydraulic analysis of tidal currents, levels, and flushing was conducted with various gate configurations to ensure minimal impacts to navigation and the environment. Details of the analysis, including model development and its calibration, is presented in the Hydrology and Hydraulics Appendix.

The proposed floodgate structure determined from the hydrologic and hydraulic analysis consists of one 100-foot wide navigation gate (invert at -18 feet NGVD) and ten 50-foot wide gates, 14 feet high (inverts at -14 feet NGVD). Due to the geometry of the channel bottom, the desire to minimize scour and shoaling and the need to provide the most efficient flow area, the gates were located in the deepest parts of the channel, away from the more easily erodible south side (see the Main Report for detailed drawings of the gate configuration).

(2) Current Analysis at the Floodgate. Six different floodgate opening configurations were evaluated. A description of their configurations and the rationales for their formulations are listed in table 13. The alternatives were evaluated for mean, mean spring, and maximum astronomic tide ranges and the results are reported in table 14. Average velocities range from 10.8 fps for mean spring tide for the FC alternative to 1.6 fps for the EN alternative. The EN alternative produces velocities that are very close to those which exist at the General Edwards bridge under a mean spring tide condition. Alternative N3 is the first alternative where average velocities for all existing astronomic tide conditions remain under the prescribed 3 knot criterion (see Hydrology and Hydraulics Appendix). The frequency of occurrence of velocities reaching 3 knots has been determined for each alternative for today's tidal conditions and is presented in table 15. As can be seen, alternatives N3, N4, and EN produce conditions which very seldom, if ever, reach 3 knots.

Removal of the abandoned I-95 highway embankment, if it were ever to occur, and deepening of the Saugus River for the Corps' proposed navigation project could cause an approximate 10 percent increase in maximum velocities over those determined for existing conditions at the tidal gate. This estimated increase is based on an analysis using dynamic modeling methods as described in the next section. The potential velocity increase is hard to define at present since it is unknown where or how many openings would be constructed through the embankment. However, it is safe to

TABLE 13

FLOODGATE OPENING ALTERNATIVES

Alternative	Flow Area Below Mean Sea Level (square feet)	Basis of Formulation
FC	1,260	Size opening to minimum required to pass navigation vessels without consideration to velocity.
N1	2,800	Size opening such that maximum average velocity during mean range is about 5.1 fps (3 knots).
N2	3,500	Size openings such that maximum average velocity during mean spring range is about 5.1 fps (3 knot).
N3	5,200	Size openings such that maximum average velocity during maximum astronomic range is about 5.1 fps (3.0 knots).
N4 (SELECTED)	8,700	Size openings such that cross sectional area below 0 foot NGVD is equal to the area of the smallest existing cross-section near the mouth of the Saugus River.
EN	12,170	Size openings such that maximum average velocity during mean spring range is as close to existing (1.7 fps) as possible.

TABLE 14

MAXIMUM AVERAGE VELOCITIES
(From Hydrologic Routing Analysis)
(feet per second)

Alternative	Mean Range	Mean Spring Range	Maximum Astronomic Range
Existing Condition	-	1.7*	-
FC	9.4	10.8	14.1
N1	5.2	6.1 (2.6)**	8.8 (4.5)**
N2	4.2	5.0	7.3 (3.8)**
N3	2.9	3.4	5.3
N4 (SELECTED)	1.7	2.1	3.3
EN	1.3	1.6	2.4

NOTES: *From gaging completed be U.S. Geological Survey

**Estimated number of hours during the approximate 6.25 hour period, from high to low tide when 5.1 fps (3 knot) average velocity criteria will be exceeded.

TABLE 15

ESTIMATED FREQUENCY OF EXCEEDANCE
OF VELOCITY CRITERIA

Scheme	Average Annual Hours V Above 3 Knots	Percent of Time V Above 3 Knots	Maximum Consecutive Hours V Above 3 Knots	Average Consecutive Hours V Above 3 Knots	Average Annual Number of Tidal Falls With V Above 3 Knots
Existing	0	0	0	0	0
FC	7363	84.0	6.5	5.3	1389
N1	1830	20.9	4.5	2.6	704
N2	291	3.3	3.8	1.0	291
N3	0.2	Nearly 0	0.8	0.4	0.5
N4 (SELECTED)	0	0	0	0	
EN	0	0	0	0	0

Notes: V represents average cross sectional velocity for existing conditions

Total number of tidal falls per year (flood and ebb conditions) is about 1403.

TABLE 16

PREDICTED FUTURE VELOCITIES
(From Hydrologic Routing Analysis)

MAXIMUM AVERAGE VELOCITY (feet/sec)

Alternative	Condition A	Condition B
N3	3.7 (6.3 fps)	4.3 (7.4 fps)
N4 (Selected)	2.4 (4.1 fps)	2.9 (4.9 fps)
EN	1.7 (2.9 fps)	2.1 (3.6 fps)

NOTES: Condition A - Maximum astronomic tide range with breach in I-95 embankment, if it should ever occur and construction of navigation project.

Condition B - Same as Condition A with additional 1-foot rise in sea level.

say that the majority of increase in velocities would be related to the removal of the abandoned I-95 embankment, rather than the dredging of the Saugus River. The present Pines River opening in the I-95 highway fill acts as a constriction, limiting inflow through the embankment during ebb tide and restricting outflow from the area behind the embankment during floodtide. Thus, if the embankment were ever opened up or removed, an increased volume of tidewater, estimated at about 10 percent of the tidal estuary volume, would be able to pass into and out of storage in the upper Pines marsh. By contrast, deepening of the navigation channel in the Saugus River will only act to reduce friction losses causing an increase of less than 2 percent of the tidal estuary volume, since the dredging that takes place is below the spring low tide level and does not increase the storage volume within the estuary's tidal range. Table 16 presents velocities for alternatives N3, N4, and EN for future conditions.

With alternative N4 adopted, maximum average velocity would always be kept at or below 3 knots, even under anticipated future conditions, imposing no significant navigation restriction. Alternative N4 also provides greater spatial flushing capability near the structure so that biological impacts resulting from velocity changes would be lessened. If some degree of occasional hindrance to navigation or biota can be accepted, alternative N3 or a compromise scheme may provide a more cost effective solution. Alternative EN keeps average velocities at present levels but at a much larger expense; alternatives FC and N1 will likely be unacceptable to the public due to relatively frequent high currents. A hydrodynamic model will be necessary in the Project Engineering and Design Phase to demonstrate that the smallest gate opening has been selected consistent with minimal adverse impacts on navigation. In addition, the spatial impact on currents will be better defined allowing for more refined estimates of sedimentation. (See Hydrology and Hydraulics Appendix, Addendum II).

(3) Hydrodynamic Analysis of the Tidal Estuary

(a) General. Aside from analyzing the effects of velocities at the floodgate, the most important part of identifying water quality impacts is the determination of the change in tidal levels, flushing and salinity characteristics caused by the tidal floodgate structure. Water quality data collected by NED and others have shown that the estuary can normally be considered fairly well mixed throughout the entire water body allowing it to be simulated, from a salinity standpoint, by a one-dimensional model as described in the Hydrology and Hydraulics Appendix.

In addition, aerial photography used in obtaining the area-capacity relationship confirms the general one-dimensionality of the estuary; flow is confined within narrow channels up to approximate mean high water level (4.9 feet NGVD), after which the water spreads out over the relatively flat saltmarsh area. Plates 3a, 3b, and 3c, present approximate water surface areas for various tide levels, obtained from aerial photography.

It should be noted in using plates 3a-3c that water depth of only about one foot among marsh grass is difficult to differentiate with aerial photography. For example, on plate 3A, interpretation of the photographs did not reveal water in the high marsh north of Saugus Raceway to above Lewis Lane when the estuary water level reached elevation 6.9 feet NGVD. Field investigations by environmentalists later revealed the area would normally be inundated for that tide level.

(b) Discussion of Results. Even though calibration of the model shows that it is generally a good hydraulic representation of the Saugus and Pines Rivers, it is extremely difficult to accurately predict minor changes in the flushing and related salinity characteristics in the small creeks and ditches of the estuary. This is due to the complexity and variability of various factors including wind-speed and direction, variation in tides, sea level changes, storms, etc. Therefore, the intent of the model application was to compare tidal motion and flushing characteristics of various floodgate schemes to the modelled "existing" condition alternative (alternative EN) for a range of tidal conditions. In this way, relative impacts of each alternative could be determined.

Hydrodynamic analyses described in detail in the Hydrology and Hydraulics Appendix were completed for various alternatives described in table 13 and results showing tidal characteristics are displayed in tables 17, 18, 19, and 20. Neither tide level nor timing changes show major differences from existing conditions for schemes N2, N3, N4, and EN. However, as a result of the velocity considerations discussed in the Hydrology and Hydraulics Appendix, scheme N4 appears to be the most promising from a navigation perspective without making the floodgate structure cost excessive.

Since tidal motion is similar to a moving long period wave it is not possible to determine the exact change in flushing volume for various alternatives by simply examining changes in tidal timing and levels; therefore, the following analysis was completed. Variable flow data, as

TABLE 17

EFFECT ON HIGH TIDES
SAUGUS RIVER ESTUARY
(Change in high tide levels
from existing condition
for selected tide conditions)
(In Feet)

Alternative	At Seaplane Basin			At Atlantic Lobster			At Lincoln Avenue		
	Mean High	Mean Spring High	Maximum Astronomic High	Mean High	Mean Spring High	Maximum Astronomic High	Mean High	Mean Spring High	Maximum Astronomic High
FC	-0.35	-0.45	-0.75	-0.30	-0.40	-1.05	-0.25	-0.40	-1.10
N1	Negligible	-0.05	-0.15	Negligible	-0.05	-0.20	Negligible	-0.05	-0.20
N2	Negligible	Negligible	-0.10	Negligible	Negligible	-0.10	Negligible	Negligible	-0.10
N3	Negligible	Negligible	-0.05	Negligible	Negligible	-0.05	Negligible	Negligible	-0.05
N4*	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible
EN	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible

* SELECTED

NOTES: 1. Water levels were determined using a one-dimensional dynamic routing analysis
2. The term "Negligible" refers to changes in values from existing conditions of less than 0.05 foot

TABLE 18

EFFECT ON LOW TIDES
SAUGUS RIVER ESTUARY
(Change in low tide levels from existing condition for selected tide conditions)
(In Feet)

Alternative	At Atlantic Lobster			At Lincoln Avenue		
	Mean Low	Mean Spring Low	Minimum Astronomic Low	Mean Low	Mean Spring Low	Minimum Astronomic Low
FC	+0.75	+1.15	+2.60	+0.70	+0.90	+1.50
N1	Negligible	+0.10	+0.20	Negligible	+0.05	+0.10
N2	Negligible	Negligible	Negligible	Negligible	Negligible	+0.05
N3	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible
N4*	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible
EN	Negligible	Negligible	Negligible	Negligible	Negligible	Negligible

* SELECTED

NOTES:

1. Water levels are determined using a one-dimensional dynamic routing analysis. The use of this method in determining low tide level is extremely sensitive to channel configurations. As a result, it is felt that exact determination of alternative impacts on the Seaplane Basin are not possible because channel topographical information upstream from Route 107 is spotty at best. However, low tide levels at Atlantic Lobster can be used as a good indication of what will generally happen at the Seaplane Basin.

2. The term 'negligible' refers to changes in values from existing conditions of less than 0.05 foot.

TABLE 19

TIMING DELAY IN OCCURRENCE OF HIGH TIDE
(Change From Existing Condition in Minutes)

Alternative	At Seaplane Basin	At Atlantic Lobster	At Lincoln Avenue
FC	39	63	55
N1	13	18	18
N2	9	12	16
N3	3	5	6
N4 (SELECTED)	0.5	0.5	
1			
EN	0	0	0

NOTE: This is for Maximum Astronomic Tide Range.

TABLE 20

TIMING DELAY IN OCCURRENCE OF LOW TIDE
(Change From Existing Condition in minutes)

Alternative	At Atlantic Lobster	At Lincoln Avenue
FC	78	61
N1	29	11
N2	18	10
N3	8	4
N4 (SELECTED)	1	1
EN	0	0

NOTE: This is for Maximum Astronomic Tide Range.

computed by the model for the station at the General Edwards bridge, was averaged between time increments, and accumulated to determine total tidal exchange volume. This volume determination was completed for the existing condition at the General Edwards bridge and gate alternatives N3 and N4. Three different tide ranges were evaluated: maximum astronomic, mean spring, and mean. These tide ranges under existing conditions in the model (alternative EN), produced 9,100, 6,400, and 5,700 acre-feet of tidal interchange volume, respectively. After simulating installation of the tidal floodgate for alternatives N3 and N4, the flushing volume decreased by a maximum of 3 percent for alternative N3 and less than 0.1 percent for alternative N4. The 0.1 percent reduction associated with alternative N4 is so small that any associated impacts on water quality, including salinity, would be nearly impossible to notice and impossible to quantify. Any reduced exchange volume associated with alternatives N3 and N4 might be at least partially recovered if other constrictions, such as narrow bridge openings on the Saugus and Pines Rivers were opened up and allowed to flush more freely or if the Saugus River navigation project was completed.

An analysis was completed to show the effect if the I-95 embankment were ever removed. The increase in flushing volume due to removal of the abandoned highway embankment was estimated to be about 10 percent higher when compared to the flushing volume for the existing condition during a maximum astronomic tide range.

An analysis was also completed assuming that the Saugus River navigation project was constructed. Through dredging, the flow area of the Saugus River would increase, reducing friction and resulting in an approximate 2 percent increase in flushing volume over the existing condition. Thus, if any of the above mentioned actions were implemented in the estuary, tidal flushing changes resulting from construction of the tidal floodgate would be reduced.

Comparison of velocities from the hydrodynamic model with those from the hydrologic model for the N3 and N4 alternatives showed similar results, with the hydrodynamic model producing a maximum velocity of about 4.4 fps (2.6 knots) and 2.6 fps (1.5 knots), respectively, during a maximum astronomic tide range. The hydrologic method discussed in the previous section produces somewhat higher velocities since there is no consideration for time of travel or dynamic effects in the storage routing. For comparison purposes, the hydrodynamic model was also used to compute velocities during a maximum astronomic tidal fluctuation at the Route 107 bridges over the Pines River and the Lincoln Avenue bridge. The velocities occurring during peak discharge were estimated at 3.1 (1.8) and 3.9 (2.3) fps (knots), respectively. This shows that currents at these upstream bridges will be less than those at the tidal floodgate for the N3 alternative and more than those at the tidal floodgate for the N4 alternative.

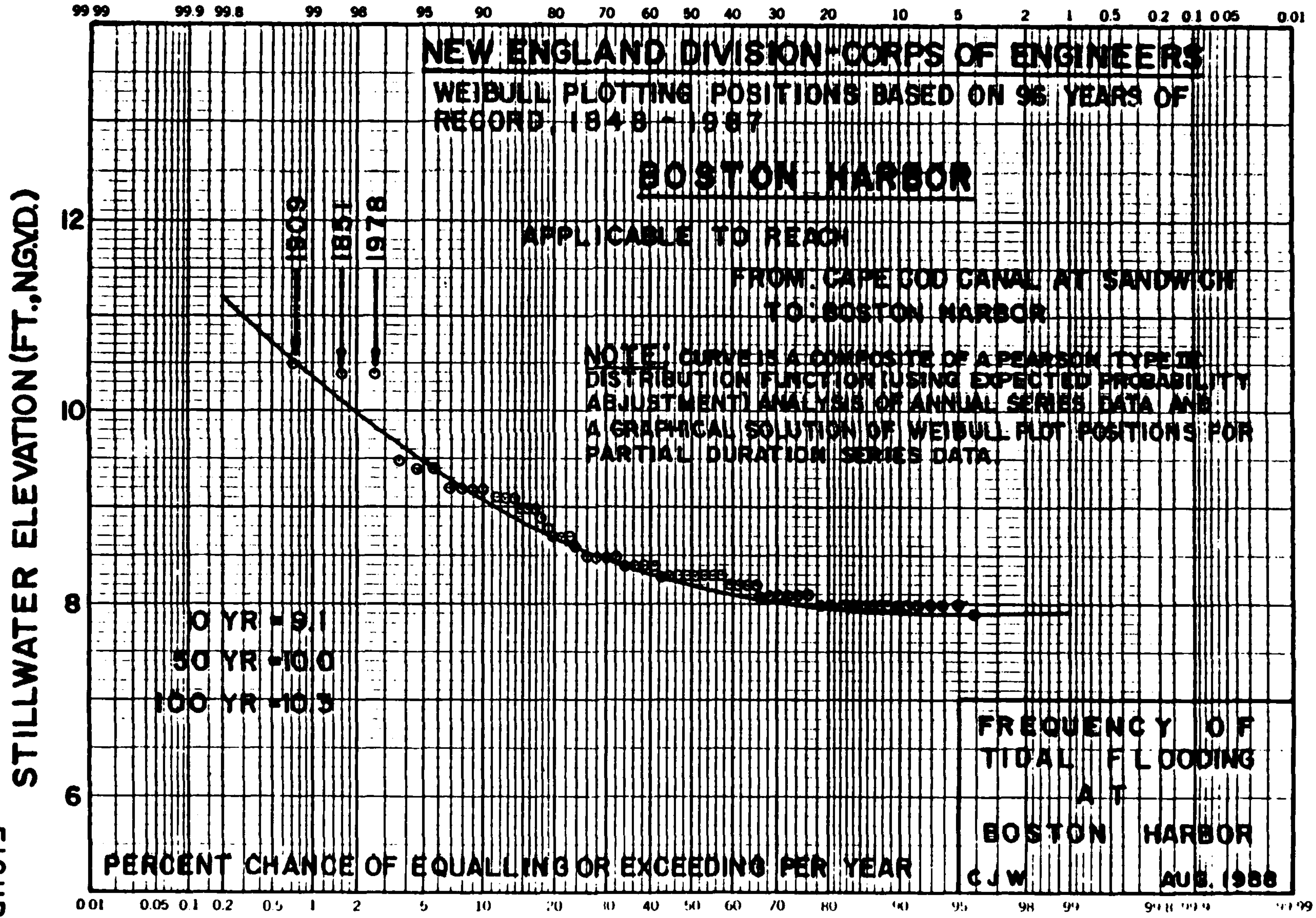
As measured during the April 1987 current observation program, peak discharge on an incoming tide occurs when the tide level is somewhere between plus 1 and 0 feet NGVD. As a result, the flushing gates preliminarily planned for this project have the top of the openings set at about 0 feet NGVD to promote their efficient use. This same configuration was simulated in the hydrologic model analysis, previously discussed. In final design, modelling may show greater efficiency in flow if the top of gate openings were set higher than 0.0 feet NGVD. Inverts (bottom of openings) were set as low as practical. (See drawings in the Main Report).

(4) <u>Discussion of Floodgate Closure</u>

(a) <u>General</u>. The total effect of astronomic tide combined with storm surge produced by wind, wave, and atmospheric pressure contributions is reflected in actual tide gage measurements. Since the astronomic tide is so variable in the study area (mean tide range of 9.5 feet), the timing of the storm surge (maximum recorded 4.9 feet) greatly affects the magnitude of the resulting tidal flood level. Obviously a storm surge of 3 feet occurring during low astronomic tide would not be as damaging as if it occurred at a tide approaching a maximum astronomic tide level. Further details on the importance of coincident astronomic tide with storm surge in producing significant tidal flooding is provided in the Hydrology and Hydraulics Appendix.

(b) <u>Yearly Operations</u>. It is important to note that both navigation and flushing gates will remain fully open at all times, except when storm tide levels are predicted to be greater than those expected to produce flood damage. Preliminary field investigations and topographic mapping indicate the beginning of flood damages to be about 8 feet NGVD. Figure 17 shows that this level is equalled or exceeded about once annually at Boston.

As an example of the current operation at a similar existing Corps project, the New Bedford, Massachusetts hurricane barrier, which has been in operation for 19 years, has been closed a total of 124 times. This is an



WITH 1 OUTLIER

FIGURE 17

average of 6 closings/year. Data on this barrier is given for informational purposes only since the Saugus River project will have its own particular operational characteristics. The New Bedford barrier is operated for hurricanes and coastal storms while a Saugus River tide gate would be operated mainly for severe northeast storms. Nonetheless, several (2 to 3) operations a year would be possible on average. Actual gate closure would be based on predicted astronomic tides, storm intensity, tidal surge, and windspeed and direction, rainfall, and freshwater runoff. Closure would normally take place at about elevation 7 feet NGVD tide level.

(c) Duration of Floodgate Closure and Coincident Runoff. During operation of the tidal floodgate, the gates will generally be closed as the high tide approaches the start of damages and will be opened as soon as the ocean level drops to the level of the estuary. Since the sinusoidal shape of the tide hydrographs in the study area is so steep, the estimated length of time when the tide level would be above 8 feet NGVD for a minor storm would be only about an hour. Length of closure for most storms would be generally in the range of 1 to 2 hours. Even the most significant storms associated with events having less than a 1 percent chance of occurrence would have closures of about 6 or 7 hours. Further discussions and examples of operation of the floodgate under severe storms is covered in the Hydrology and Hydraulics Appendix.

An important point in the floodgate closure evaluation is the concern over the impacts of a potential change in salinity on estuarine vegetation. Closures of more than a few hours will occur only when there is a major coastal stormtide. As noted in the Hydrology and Hydraulics Appendix, major coastal storms which affect the East Coast of Massachusetts are generally "Nor'easters" which predominantly take place during late fall, winter and the early spring months when vegetation within the estuary is more or less in a dormant state. Vegetation existing under this condition is less likely to be harmed than if closure occurred during the growing season.

Historically, there have been very few high runoff events coinciding with high storm tide events. It may be that a large number of storms that do occur happen during the winter season when precipitation is in the form of snow. An analysis was made of data from a nearby USGS riverine gage at Middletown, MA on the Ipswich River and a comparison made with storm tides which were recorded at the Boston NOS gage (see plate 4). As can be seen, historically for this representative drainage area, a runoff having a 10 percent chance of occurring when tides are high produced a peak flow of only 550 cfs. Assuming the floodgate was closed for 6 hours, the volume would amount to approximately 270 acre-feet or a rise in interior estuary flood level of approximately 0.3 foot with the interior pool initially at 7 feet NGVD. Further analysis of runoff data is presented in the Hydrology and Hydraulic Appendix.

c. Discussion of Water Quality Changes

(1) General. As discussed previously, with the floodgates open, minor changes in flushing and tide levels should produce negligible changes in water quality including salinity concentrations in the Pines and Saugus River over that which occurs presently. In addition, tide levels and ranges are so naturally variable that a set pattern of flushing does not now exist; tides vary not only seasonally and monthly but daily. Under existing conditions, the estuary can experience major flushing action caused by higher than normal spring tide ranges during part of a month followed by average or less than average flushing due to mean tide or less than mean tide ranges during the rest of the month. Rising sea level, removal of constrictions within the estuary or possible dredging projects could help offset any minor loss in flushing due to the floodgate project.

A change in water quality could temporarily result when the floodgate is closed. However, this change will not be significant based on a number of factors: (1) only on rare occasions will there be a high amount of freshwater runoff coinciding with a significant storm tide, (2) even if there is a high runoff rate, the length of time when the gates would be closed would be short, generally less than 3 to 4 hours limiting the total volume of runoff during closure, (3) there are no significant point source wastewater discharges into the Pines and Saugus Rivers which would cause harm to the estuarine river system before the gates would be opened again and normal tidal flushing resumed, and (4) storm induced wave action and tidal currents would continue to mix the estuary waters.

(2) Impacts from Major Pollution Sources. Water quality will more likely be affected by other changes in and near the estuary rather than by construction of the floodgate structure. For instance, removal of Lynn's combined sewer overflow (CSO) or the capping and closure of the Saugus landfill will have a much greater impact; probably more than any increase in nonpoint source pollution which could result from increased runoff associated with further watershed

urbanization. Also, after the Lynn Water and Sewer Commission (LWSC) wastewater treatment plant (WWTP) changes from primary to secondary treatment, there will be less chance of WWTP pollutants, including some nutrients, from being accumulated within the estuarine waters. The impacts resulting from CSO removal, the Saugus landfill closure scheme and improvements to the Lynn WWTP are presently being evaluated in ongoing studies by others.

At the present time it seems likely that the CSO at Summer Street will be eliminated through the separation of sewers from storm drains in the next few years. This removal will not only reduce the number of times that state coliform criteria are exceeded but will reduce the amount of organic and toxic buildup in the sediments near the overflow. However, since CSO removal is yet to be implemented, water quality parameters were evaluated on the assumption that the CSO would remain. Average currents and tide stages were determined from the hydrodynamic model for the Saugus River near the CSO location and show imperceptable changes when comparing existing conditions to proposed conditions; dilution and flushing of CSO pollutants during an open gated condition will not change.

When gate closure does occur, the first flush of the CSO (the most concentrated pollutants) would usually have already occurred since it takes a major storm system to cause closure and rainfall would generally precede the storm surge. If on the rare occasion the first flush of a combined sewer overflow event occurs coincident with closure, the dispersion of the pollutants will not act much differently than under existing conditions when the first flush occurs coincident with a slack high tide period. Storm induced wave and current action in the estuary would contribute to continued mixing and dispersion of pollutants, as presently occurs. After floodgates are opened, normal flushing characteristics of the estuary will return, distributing pollutants as before within a few tide cycles.

Proper capping of the landfill to eliminate any leaching of metals seems probable since not only has there been a state ordered study to identify and solve existing problems but a more recent study is evaluating the use of the landfill for an MBTA parking facility. Water quality parameters for future conditions were evaluated on the assumption that improved capping procedures have yet to take place. The Corps will continue to closely monitor these efforts and provide coordination as applicable.

It is anticipated that thermal releases from General Electric, RESCO and Eastern Tool Company will continue to be made. Even though tidal flushing of the estuary with the gate open will be slightly changed, the locations of thermal discharges are close to the voluminous portions of the estuary and the floodgate structures, so changes to the dilution characteristics of the thermal releases are expected to be minimal.

Individual water quality parameters are described in the following sections. It should be noted that for all the water quality parameters discussed, it is assumed that no significant development would take place in the existing wetland area (see Environmental Impact Report). This is the area which is presently flooded during maximum spring high tides. Deviation from this assumption would require a complete reanalysis of not only the hydraulic conditions governing this project, but the related water quality parameters.

(3) Salinity

(a) General. None of the alternatives with the exception of the Regional Floodgate Plan will produce any changes in the salinity levels within the estuary. The freshwater flow component of the 47-square mile Saugus River Basin, in general, is minor compared to the total flow which passes by the General Edwards bridge at the mouth of the Saugus River. Average annual flow for the basin is estimated at 80 cfs and runoff events having a 50 and 10 percent chance of occurrence during any particular year have peak daily flow rates estimated at 850 and 1,900 cfs, respectively. In comparison, average flood and ebb flow rates for a mean tidal range fluctuation are estimated at 9,400 cfs which is about five times the 1,900 cfs flow rate. Table 21 summarizes the saltwater-freshwater flow rate at the General Edwards bridge. For the Regional Floodgate Plan, the open and closed gate condition must be evaluated separately since the factors relating to salinity are so different.

(b) Impacts with an Open Floodgate. As described previously, the majority of the estuary can be considered well mixed with insignificant variation in vertical salinity in the area downstream from the Route 107 bridges over the Saugus and Pines Rivers. The major reason for this condition is that average annual freshwater runoff for the Saugus River (including the Pines River) is not significant, the average flow of 80 cfs amounting to approximately 160 acre-feet of volume over the course of one 24-hour day. This volume of freshwater is dwarfed when compared to a single

TABLE 21

SALTWATER-FRESHWATER FLOW RATIO
(ESTIMATED FOR SAUGUS RIVER
AT GENERAL EDWARDS BRIDGE)

Freshwater Peak Flow Rate (cfs)	Percent Chance of Occurrence	Average Tidal Flow Rate for Mean Tide Range (cfs)	Saltwater Freshwater Flow Ratio
850	50 (2-year)	9,400	11.1
1,900	10 (10-year)	9,400	4.9
3,300	2 (50-year)	9,400	2.8
4,000	1 (100-year)	9,400	2.4

NOTES:
1. Average daily freshwater flow rate is equal to 80 cfs.
2. Peak freshwater flow rate does not necessarily have a duration as long as a 6-hour tide cycle. It is shown only for comparison purposes.
3. Average tidal flow rate was determined by dividing the volume of water entering the estuary during a mean tide cycle by 6 hours.

12-hour mean tide cycle fluctuation during which approximately 4,900 acre-feet of saline water are interchanged. A 0.1 percent change in flushing volume due to the floodgate will produce immeasurable salinity differences in this area.

Upstream from Route 107 on the Pines River near the Town Line Brook tide gate, there was only minor variation in salinity noted from the top to bottom of the water column during a nearly average runoff condition on 16 August 1986 (50 cfs). Salinity remained high even at low tide throughout the Pines River's entire length indicating for these conditions that the freshwater constituent is minor.

The upper Pines River is impacted by the flow constriction in the abandoned I-95 embankment which does not allow the ponded area upstream from the embankment to drain freely. This was first noted during tide measurements made upstream and downstream of the embankment on 18 May 1987. A value of -3.0 feet NGVD was measured at the Town Line Brook tide gate and -5.0 feet NGVD was measured at the Fox Hill drawbridge at dead low tide. Visual inspection of the I-95 constriction shortly thereafter during another low tide condition revealed that the constriction acts as a dam at extreme low tide (tides below elevation -3.0 feet NGVD), not allowing the upstream area to drain completely. It should also be noted that the volume associated with a 12-hour mean tidal fluctuation upstream from the I-95 embankment is approximately 750 acre-feet while that associated with the freshwater component of an average annual flow is about 25 acre-feet. Since the freshwater component is so small in comparison to saltwater volume, and since peak high and low tide level changes due to the floodgate are insignificant, a change in flushing volume estimated at less than 0.1 percent would produce immeasurable salinity differences.

On the Saugus River, between the Route 107 bridge and the Lincoln Avenue bridge, only minor top to bottom variations in salinity have been measured. The saltwater volume interchange in this area has been estimated at approximately 750 acre-feet for a 12-hour mean tidal fluctuation, while the freshwater component for a 12-hour average annual runoff condition has been estimated at approximately 75 acre-feet. Even though the freshwater component is a slightly greater percentage of the total flow, this area is much more turbulent due to narrow bridges and bends than that in the upper Pines River. Therefore, a flushing volume change of less than 0.1 percent will create a change of less than 0.01 ppt in salinity concentrations in this well mixed area. Salinities are estimated to range from 25 to 27 ppt during high tide at the Fox Hill bridge. Therefore, this change is meaningless compared to the variability which exists in flushing volumes, tide levels, freshwater runoff volumes, and ocean salinity concentrations.

The portion of the estuary which could be most influenced by the floodgate is the Saugus River above Lincoln Avenue and Shute Brook. The freshwater component makes up a larger portion of the total volume, and there is less turbulence in these areas. The saltwater flushing volume for a 12-hour mean tide fluctuation is about 190 acre-feet. Assuming a freshwater discharge of 80 cfs (equivalent to the average annual flow), the total freshwater volume would be 75 acre-feet or about one-third the saltwater volume. In general, a 0.1 percent reduction in flushing volume would not be considered significant, since as shown in the hydrodynamic modelling, changes in high and low tide elevations are negligible. However, since the freshwater portion is such a major component of the flow, further analysis was completed.

Assuming that a maximum reduction of 0.1 percent of 190 acre-feet (about 0.2 acre-feet) of tidal flushing volume occurs when the floodgate is put in place and that there is no mixing of freshwater with saline water, the saline water will remain near the bottom as it travels upstream during high tide. The reduction in salinity layer thickness or depth will be less than 0.01 foot. The wetland area that would have been in direct contact with a 0.01 foot thick horizontal "slice" of saline water before floodgate construction would be approximately 0.1 acre.

The above discussion provides a conservative estimate since the physical movement of a salt wedge does not behave in this simplistic manner. First, as shown by the hydrodynamic model, peak tide levels with the floodgate do not change from existing conditions; therefore, saline waters will reach the same levels as before, only not for as long a period. Secondly, the horizontal slice representation does not occur in nature since frictional forces tend to keep saltwater confined more toward the center of the channel with a mixture of freshwater and saltwater flowing along the edge of the channel. This condition was documented during field measurements that were made in this area on 1 and 2 December 1986. Also during the same measurements, it was noted that all saltwater species on the upper Saugus River and Shute Brook were inundated by at least several tenths of a foot of water when the high tide levels reached 6.5 and 6.8 feet NGVD. This inundation will continue with the proposed floodgate in place. Impacts related to a hypothetical 0.01 foot thick salinity layer reduction would not only be decreased by the above mentioned factors but also by the variability of flushing volumes, tide levels, freshwater volumes, and ocean salinity concentrations.

Other factors may compensate for any lost tidal flushing volumes on both the Saugus and Pines Rivers. Sea level rise if it proceeds at the historic rate of record would offset this loss of salinity within a few years. Navigation dredging of the Saugus River would immediately offset the flushing volume lost on the Saugus River. The possible, but unlikely, I-95 embankment breaching could make significant changes in flushing volumes on the Pines River. As well, the enlarging of narrow bridge openings could increase tidal flushing.

(c) Impacts with Floodgate Closed. As discussed in the floodgate operational procedures described in the Hydraulics and Hydrology Appendix, floodgate closures are expected to be infrequent, taking place on an average of 2 to 3 times a year with an average closure time between 1 to 2 hours. Even with a major tidal event the maximum length of time the gates will be closed would be about 6 or 7 hours.

As shown in plate 4, historically, the chance of having heavy runoff (greater than an average daily flow of 500 cfs), occurring at the time of a major tidal flood event has been rare. It is likely that the coincident type of events displayed in this graph will continue.

In addition, it should be noted that most closures will occur during the nongrowing season, November through March, since that is when significant northeast storms generally occur. These storms are of the extratropical variety, generating off the coast of Georgia or the Carolinas and traveling northward along the coast. Temperatures at this time are cold and precipitation has a good chance of being snow. Significant storm surges occurring along the east coast of New England during the summer and early fall are generally the result of tropical storms (i.e.,hurricanes). The occurrence of these storms in association with resultant storm surges that effect the east coast of Massachusetts at the project area are very rare.

Other factors should be considered when reviewing changes to salinity with the floodgate closed. For instance, if a significant storm did occur and the gates are closed longer than the normal closure period of 1 to 2 hours, high wind and wave conditions will be such that the water body will be well mixed with little or no salinity stratification taking place. Also, if the event is minor, closure will be for only 1 to 2 hours and salinity conditions during closure should not be much different from what presently occurs with high runoff coincident with slackwater around high tide.

Closure will take place when it is expected the tides will reach or exceed about 8 feet NGVD. If there is very little runoff expected and no problem with freezing temperatures, severe winds or snowbanks (drainage), closure will take place about 7.0 feet NGVD. Maximum astronomic high tide events up to about 7.5 feet NGVD without a coastal storm will continue to occur. All tides over 8 feet will be eliminated. Because the occurrence of tide events above 7.5 feet NGVD are relatively infrequent, on an average of 3 times per year, the change of salinity in the upper areas of the estuary will be negligible.

(4) Coliform Bacteria. The construction of a tidal floodgate will have an imperceptible impact on existing coliform bacteria levels when the gates are open and only a slight effect on the dispersion of the bacteria when the gates are closed. For the open condition, there will be no change in the numbers of bacteria released to the estuary and minimal if any change in the major criteria which affect bacterial growth (i.e., organic loading, temperature, salinity and dispersion characteristics). As discussed previously, during dry weather, high bacterial levels in the upper parts of the estuary, generally above the Route 107 bridges, are diluted by movement of the tidal prism such that during high tide, Massachusetts coliform criteria are not exceeded in the lower basin below the Route 107 bridges. Wet weather conditions increase the problem when flows emanating from Lynn's combined sewer overflow (CSO) and street drainage from the lower basin combine with failing septic systems from the upper basin to cause water quality exceedances throughout the estuary.

Bacterial loading from the CSO and street drainage is extremely variable depending on the amount of antecedent rainfall, intensity and duration of the storm, seasonal variations, etc. This variability which can cause a magnitude of difference in bacterial loading makes an analysis of a 0.1 percent change in flushing volume essentially meaningless. With the gates open, tide levels do not change, and current changes throughout the estuary (with the exception of the area adjacent to the barrier) are imperceptible. Therefore, bacterial dispersion will not behave any differently than it does now. Bacteria reduction through die-off will also continue as it does now since growth factors will not have changed.

When the floodgate is closed, there will be a significant change in the currents, interior water levels, and possibly some minor changes in salinity, temperature, and organic loading depending on the amount of freshwater runoff which occurs during the gate closure period. Analysis of historic runoff patterns have indicated that major runoff coincident with high storm tides is an infrequent event. Bacterial as well as organic loadings that once were dispersed by movement of the tidal prism (essentially the elongation of the freshwater plume in the direction of the riverine current) will now be dispersed into a slackwater condition. Because of the lack of currents, solids entering from a riverbank outfall will not spread out as far, concentrating both the bacteria and organics towards the center of the channel rather than along the edge. Bacteria entering from the upper riverine and brook portions of the basin will show a reduction in the length of dispersion temporarily since there will be no seaward currents. The concentration of bacteria and organics in a smaller plume size may increase bacterial growth. However, it is felt that this increase will be minor since existing average currents throughout the estuary are generally very small at peak tide conditions (less than 1 fps). Also, the closure will be very short (generally less than 3 to 4 hours) and infrequent with flushing action returning to normal soon after. With significant freshwater runoff, there may also be some changes in salinity and temperature during times of closure. But again, changes in bacterial growth should not be significant since gate closure will be limited to only a few hours.

Regardless of whether or not the floodgate is built, bacterial loading from man's activities will continue to limit harvesting of the potentially valuable shellfish resource within the estuary. Changes in bacterial dispersion and growth resulting from operation of the floodgate will be minor. If completion of Lynn's CSO study results in the removal of Summer Street overflow, the implementation of improved basin management practices which eliminate other nonpoint sources of contamination, may result in the restoration of a useable shellfish resource.

(5) Dissolved Oxygen and BOD. Continuation of high organic and nutrient waste loading along the crowded coastal communities from Massachusetts to the Carolinas has been cited by numerous State and Federal agencies for causing the most prevalent water quality problem to affect the East Coast--dissolved oxygen depletion. Nutrients and organics from LWSC primary wastewater treatment plant, Lynn's Summer Street combined sewer overflow, and other nutrients from nonpoint sources within the basin have created conditions where, at certain times of the year as a result of decomposing wastes and prolific algae growth, dissolved oxygen levels have not met State standards for class SB waters. The slight reduction in flushing within the basin due to the project

will reduce the amount of tidewater to come in contact with wastes that are discharged into the estuary and as such could theoretically reduce the DO slightly during times of CSO release, however, any change, as shown in the analysis later in this section, will be immeasurable. During warm summer weather when there is rapid algae growth and subsequent decomposing algae conditions, a slight increase in DO depletion will occur if there are few storms to churn up and aerate bottom waters. This condition is not expected to change due to the project.

Because of the variability which exists with regard to tidal stage, tidal currents and their directions, intensity and length of rainfall, and seasonal conditions, it is difficult to determine the exact impact of any combined sewer overflow on water quality within the estuary. The consultants for the Lynn Water and Sewer Commission have been involved in using a two-dimensional mathematical model to define impacts of the CSO's on the Saugus River and Lynn Harbor. However, recently they have placed less emphasis on analyzing the CSO's impact on the Saugus River since their recommended plan calls for the separation of the sewers from the storm drains eliminating almost the entire pollutant loading from the Saugus River.

Until results of that study are finalized, some type of analysis was necessary to estimate the differences associated with the BOD loading under existing conditions versus those with construction of a tidal floodgate. A riverine-type analysis solving for dissolved oxygen depletion using the Streeter-Phelps equation was not applicable for this area because of multidirectional aspects of the tidal movement within the Saugus River. Instead, the movement of the tidal flow was used to drive and distribute the organic load within the estuary. For purposes of analysis, peak tidal floodflow and ebb flow as determined at the Summer Street CSO site from the hydrodynamic model for a mean tide range condition, are estimated at 1,700 and 1,550 cfs, respectively, while average annual freshwater flow is estimated at 80 cfs.

A simplistic approach was used to estimate impacts of heavy organic loading from the CSO on the Saugus River. An average overflow event was determined assuming: a BOD loading of 2,400 lb/event (95,000 lb-BOD/year evenly distributed over 40 events/year); tidal flow conditions which distribute loadings completely in the ebb direction or completely in the flood direction; and organic material would be mixed with the flow stream, deplete DO as the slug moves upstream, and not settle out until the BOD is depleted. It was also conservatively assumed that there would be no reaeration to replenish the DO.

Under the existing conditions, the length of reach before BOD would be used up was determined to be about 7,600 linear feet (LF) in the flood direction and about 2,100 LF in the ebb direction. The difference in length is attributable to the volume of water which exists upstream and downstream of the site. A decrease in flushing volume from installation of the floodgate, estimated conservatively at 0.1 percent, would increase the length of DO depletion by less than 25 LF in the flood direction and less than 10 feet in the ebb direction. This change is not considered significant when looking at the total length affected by low DO or even noticeable when looking at the variability of dispersion characteristics, tidal range fluctuations, and the quantity of organic loadings.

All other sources of organic loadings are not point sources (e.g., flow from upstream basin, street drains, overland flow, etc.) and as such it would be difficult to determine differences related to the floodgate. However, since the amount of dissolved oxygen available is directly related to, among other factors, the volume of water (i.e., DO content) which comes in contact with organic material, it can be conservatively stated that a 0.1 percent change in flushing volume with gate scheme N4 will result in a 0.1 percent change in dissolved oxygen levels. Again this can be considered insignificant when compared to the amount of reaeration which will take place, the quantity of waste loads, temperature, tide levels, etc.

Flushing changes from sea level rise, enlarging narrow bridge openings or implementation of the Saugus River navigation project, would negate any flushing loss associated with the floodgate project. DO levels would at least revert back to existing conditions and may actually increase. Cleanup of the Saugus River through removal or treatment of combined sewer overflows and through implementation of land use controls to control nonpoint sources (i.e., more frequent street sweeping, controlled use of lawn fertilizers, etc.) would more than offset any impact the floodgate would have on dissolved oxygen levels. It should also be noted that in 1988, construction is beginning for the conversion of the LWSC wastewater treatment plant located in southeast Lynn from primary to secondary treatment. This project will remove significantly more organic material from the receiving waters, improving the quality of the flushing water at the mouth of the Saugus River.

During infrequent events when gates are closed and there is a heavy runoff condition, organic sediments may not spread out as far. However, after the gates are opened, the material will in general move to where it would have gone under natural flushing conditions. DO depletion will not be a problem with the gates closed since closure is for only a few hours.

(6) Nutrients. In general, with a change in tidal flushing volumes, nutrients from point and nonpoint sources will become more concentrated in the estuary water columns and; therefore, could make the estuary more susceptible to algae bloom conditions. However, a minor flushing change of 0.1 percent with gate scheme N4 will change the nutrient concentration in most of the basin by the same magnitude thus having an insignificant impact. The upper ends of both the Saugus and Pines Rivers estuaries would receive approximately the same reduction in flushing volume. However, because of hydraulic restrictions and reduced total volume available, there is less kinetic mixing action and these areas would be more susceptible to minor increased algae bloom conditions. Sea level rises, the opening of narrow bridge openings, and the implementation of the Saugus River navigation project would decrease nutrient concentrations through increased flushing, thereby reducing concerns over algae growth. Gate closure will result in reduced dispersion of nutrients; however the condition will last for only a short time before flushing action once more resumes.

Analysis of nutrient levels (nitrate-nitrogen and total phosphorous) indicates that algae growth would be nitrogen limited based on the stochiometric requirements of algae's N:P ratio. It was also noted that with the present total phosphorous concentration in the estuary measured during a dry weather period (average concentration equals 0.07 mg/L), there are in general insufficient amounts of phosphorous available to produce algae bloom conditions in the fully flushing Saugus River estuary (0.10 mg/L P is required for an algae bloom condition in a rapidly moving water body).

Average ammonia levels of 0.18 mg/l, although not very high, indicate that there is some decomposition of deposited organic matter taking place. This can be related to bacterial action on settled material that is supplied not only from CSO loading but from nonpoint sources such as street drains, and possibly even the Saugus landfill, where leachates may be entering the estuary from previously deposited organic wastes. It has been noted by State officials that leachate releases are still visible when high tides retreat from the inlets surrounding the landfill. It is expected that any deficiencies in the capping procedure will be identified by RESCO's consultant and corrected so that future nutrient releases from the landfill will be minor.

Release of ammonia from various sources may explain some of the reasons for the low dissolved oxygen levels which have been measured in the sluggish upper reaches of the Saugus and Pines Rivers. As decay of nitrogenous organic wastes take place, bacteria first produces ammonia, which then through oxidation reaction is changed to nitrates depleting DO in the process.

Wet weather flow conditions were analyzed by LWSC consultants to ascertain the impacts of high nutrient loadings from the Summer Street CSO. From this analysis it appears that nutrient and organic loadings provide only temporary problems; whereas, toxic releases at the time of overflow and from sediments built up from CSO releases may provide long term problems. Separation of the sewers from the storm drains as is planned will reduce the nutrient problem.

(7) <u>Suspended Solids and Sediment Movement</u>. The Regional Floodgate Plan will result in little or no change in the sediment and suspended solids movement within the estuary; the one exception being the area within a few hundred feet of the gate structure.

As discussed previously, presently, there is insufficient energy to transport significant solids loads from freshwater portions of the basin during dry weather conditions. Most solids movement takes place during the infrequent major storm events. Typical grain sizes of sediment within the estuary confirm this fact since most particles are small as is shown in the sediment boring data for the Saugus and Pines Rivers navigation studies (see Hydrology and Hydraulics Appendix). The grain sizes range from fine organic silt to coarse sand with the finer particles located generally in the upper estuary and the coarser particles located closer to the General Edwards bridge. From reviewing the boring data, the highest tractive or sediment moving force is located near the center of the channel and near the confluence of the Saugus and Pines Rivers.

Discharge from Lynn's Summer Street CSO appears to be causing a buildup of fine silt and organic sediments, rich in chromium, lead and zinc, in the area immediately upstream and downstream from its discharge point, generally in the area between Route 107 and the Lincoln Avenue bridge on the Saugus

River (see sediment sampling descriptions for A to D and J to M in the Hydrology and the Hydraulics Appendix). Fine materials which pass the 200 millimeter screen have concentrations generally greater than 50 percent of the total sediment in this area. Lynn Water & Sewer Commission's CSO consultants have recommended that the combined sewer be separated in this area thus eliminating this problem in the future and improving the overall water quality within the estuary.

Changes in velocities occurring at the mouth due to floodgate construction will be dampened out through friction and restrictions as one proceeds upstream into the upper estuary (above Route 107). Therefore, with a slight reduction in flushing volume (0.1 percent with gate scheme N4) associated with the open gated condition under a floodgate plan, solids and suspended solids movement in the upper estuary will tend to decrease only slightly since a 0.1 percent change in a small ebb flow velocity (less than 1 fps generally) will be imperceptible. The change in sedimentation is expected to be immeasurable. In comparison, the navigation project planned for the Saugus River will somewhat reduce main channel velocities in the area upstream from Route 107 where dredging is proposed due to the increase of the flow area. Increased sedimentation in the dredged channel upstream of Route 107 resulting from the navigation project is expected to be more significant than that occurring from the installation and operation of the tidal floodgate.

Sediment flushing upstream from the I-95 embankment will remain essentially as it is now if the embankment is not opened up. Sea level rise will cause some increase in sediment movement through increased flushing.

Overall average velocities in the area between the General Edwards Bridge and Route 107 will remain the same; therefore, there is no concern over causing any more erosion than under existing conditions of the portions of the riverbank enclosing the Saugus landfill. Changes in flushing caused by sea level rise and implementation of the Saugus River Navigation Project may somewhat increase velocities along the landfill.

In the area near the floodgate structure, solids and sediment movement are more complex. Riprap placed along the bottom and sides will be used to prevent scour in the area adjacent to the floodgate structure and should reduce excessive soils movement resulting from the restriction. Localized velocities will be dissipated within a few hundred feet of the floodgate structure so that sediment scour outside the riprapped area should be minor. Sediment accretion will change; however, within a few hundred feet of the floodgate with increased settling taking place along the riverbanks where velocities will be reduced due to the backwater caused by restriction of the channel. More detail on areas of increased shoaling and erosion is given in the Hydrology and Hydraulics Appendix. If the project continues into the Project Engineering and Design Phase, a model will be necessary to analyze the localized velocities near the floodgate and to identify problem shoaling areas.

Sediment buildup on the ocean side of the floodgate could increase only slightly more than it does now, since there is only a small reduction in outward moving flushing volume. These factors are not expected to change the northerly progression of sediment from the Revere Beach-Point of Pines littoral zone.

When the floodgate is closed, some fine particle deposition should take place within the estuary as the estuary's tidal movement is interrupted. Runoff from upstream areas, nonpoint sources and the Summer Street CSO will contribute solids to the system which will settle out under relatively quiescent conditions which should last under the most severe conditions for up to 6 to 7 hours. Although tidal motion will be interrupted during this period, strong storm winds and resulting waves should promote some continuing resuspension of solids. After several days, there should be no change in sedimentation since the material that would have been entrained under existing conditions will be reentrained during the following tidal cycles and carried downstream to the ocean through natural flushing.

One condition which will result from the installation and operation of the floodgate is that large storm tides will no longer be allowed to periodically flush sediment from within the estuary. It is hard to quantify this impact; however, over the long term it could make the cleanup of point and nonpoint sources all that much more important. Polluted sediments which were once moved and possibly even carried out to sea by storm events will now remain in the estuary since storm tides having elevations greater than 8 feet NGVD will be eliminated from the estuary through operation of the tidal floodgate. These events although occurring quite infrequently (less than a 10 to 20 percent chance of occurrence for each year) may have played a significant role in distributing polluted sediments out into Broad Sound. It should be noted that distributing pollutants, even under existing conditions, is not an ideal situation. More emphasis should be placed on removal of pollutants through reduction at their source.

(8) Turbidity and Color. Over the long term, neither turbidity or color levels, which are generally low, are expected to change as a result of the installation of the floodgate. The minor change in flushing volume will actually reduce velocities (except in the area adjacent to the floodgate structure) thereby decreasing turbidity. Color is related to upstream freshwater flow, benthic releases, and discharges from the CSO and other nonpoint sources, so the minor decrease in flushing may increase color but only slightly above the average 10 SU estuary value.

Short term releases of turbidity may result from construction of the floodgate but the latest silt reduction techniques will be employed to minimize the impacts.

Short term releases of turbidity would also be associated with the LPP option, as dikes would be constructed along the river. Again if this option is selected, silt reduction procedures will be implemented.

If control of silt and organic releases from the Summer Street CSO or from other nonpoint sources is implemented, high color and turbidity levels will be reduced during large runoff events.

(9) Metals. Any change in metals released into the water column of the estuary as a result of the project will be insignificant. The flushing volume change is minor, and even though there is a reduction in flow, the areas that are wetted under normal astronomic tides for existing conditions, will remain so under proposed conditions, since the tide level changes are so small (less than 0.05 foot). Metals released from sediment through benthic action will still be released and flushed as before. Since there will be minimal changes in DO in the water column, anaerobic action within the sediments should not increase over what there is now.

Velocities will not increase near the landfill due to the floodgate since the locally higher velocities which occur near the floodgate restriction during the open gated condition will not carry beyond the Boston & Maine railroad bridge (the downstream boundary of the landfill). Velocities may actually be reduced due to the slight decrease in flushing so that marsh muds which have retained metals near the landfills edge will not be disturbed. Modeling to be done in the PE&D phase will verify the significance of any change in velocities near the landfill.

Future cleanup actions aimed at improving the capping of the landfill and reducing CSO discharges will reduce the release of metals under both existing and proposed conditions. Increased use of controls over nonpoint sources (increased street cleaning, etc.) will reduce metal levels still further.

In the infrequent instances when closure of the floodgate is necessitated, high runoff conditions may cause releases to be made from both point and nonpoint sources. Solids that settle out during slack-water conditions will be reentrained during the next tidal cycle when the gates are opened. The dispersion pattern of sediment containing metals will be altered during gate closure but should return to the same pattern within several tidal cycles. The elimination of damaging tidal flooding events (greater than 8 feet NGVD) will reduce the amount of leachates that are released from the Saugus landfill during those events.

Increased development in the watershed may increase organic and inorganic loadings including metals as materials which once found their way at a slow rate through natural drainage paths, are delivered more rapidly due to man's construction of streets and driveways and installation of storm drains. It is expected that this development would take place whether or not the floodgate is installed.

Sea level rise and construction of the Saugus River navigation project will increase the flushing action beyond that which exists now even if the floodgate is in place. Improved flushing will increase DO and reduce metal releases which result from anaerobic action.

(10) pH. The change in flushing action associated with the floodgate should have negligible impacts on the generally good pH levels within the estuary, since the highly buffered saltwater will continue to inundate the same areas as it does now and almost to the height it does now. Low pH conditions will continue to affect the upper estuary during times of low tide, since any organic acids emanating from upstream wetland areas or the acid rain conditions show no signs of decreasing. With the floodgate closed, parts of the tidal area above 8 feet NGVD will no longer be periodically inundated during storm events. This will be so infrequent that the reduction in the buffering action of saltwater on these areas will have negligible impacts on the overall pH levels within the estuary.

(11) Oils and Grease. The minor change in flushing action associated with the floodgate should have no impact on the oil and grease levels in the estuary . Most oil and grease releases come from the marina activity within the harbor, while other intermittent discharges occur as a result of heavy urban runoff from city streets, and from accidental discharge due to industrial accidents (GE has reported several spills). In any event, most releases are made in the lower part of the estuary (below route 107) where a small change in flushing action (0.1 percent for gate scheme N4) for the huge tidal interchange should have no impact. Increased boating activity associated with the Saugus River navigation project may increase oil and grease levels, but this increase will occur whether or not the floodgate plan is implemented. There may also be increased oil and grease levels if the harbor is used as a refuge for vessels during coastal storms. This condition would be so infrequent, the impact will be negligible. Also, the harbor will be flushed within a few tide cycles after gate closure.

(12) Temperature. The large permitted thermal releases by General Electric and RESCO make this an important parameter when considering impacts of the floodgate. With the gates open, the thermal discharge should have negligible impacts since any releases that are made occur in the lower area of the estuary where small changes (0.1 percent for gate scheme N4) in flushing will not be significant when considering the total volume of that part of the estuary.

Thermal releases have the most impact at times when there is minimal tidal exchange (neap tides). As determined from the modeling study, the smaller the ocean tide range and resulting tidal interchange volume, the less impact the floodgate has in reducing tidal fluctuation in the estuary. This is indicated by the flushing volume comparison for mean, spring and maximum astronomic tide ranges which show flushing volume changes from existing to proposed conditions ranging from no change at mean tide range to 0.1 percent difference at extreme high tide range. At minimal neap tide range fluctuation, the most critical time when the volume of heated effluent water makes up the greatest percentage of the total volume within the estuary, there will be no change in flushing volume.

For purposes of this study, it is conservatively assumed that there is a 0.1 percent difference in tidal flushing when there is a minimum astronomic tide range condition (approximately 5 feet of tide level fluctuation). Under an ebb tide condition, this amounts to a 3 mgd reduction in a 3,000 mgd flow rate at the General Edwards bridge with an extreme neap tide range. This change is minor considering the fact that the maximum volume of heated water that would generally be discharged would be 100 mgd (40 mgd from General Electric and 60 mgd from RESCO). Assuming the effluent is 20° Fahrenheit above ambient estuarine water (as allotted under the existing permits) and assuming complete mix conditions which would exist near the confluence of the two rivers under ebb tide conditions, the increase in temperature of future over existing conditions would be less than 0.001° Fahrenheit.

During a floodtide condition for the same tide range, there will not be as large a volume of water moving up the Saugus River as there will be moving in the downstream direction during ebb where the Saugus and Pines Rivers join. The flow rate moving up the Saugus River is estimated at 550 mgd. With a 0.1 percent change in flow rate brought on by the installation of the floodgate, the change in temperature is less than 0.01° Fahrenheit.

The most critical condition to affect temperature would occur when the floodgate is closed. However, there are a number of factors which may reduce the magnitude of the impacts. The heated plume will not spread out as quickly during floodgate closure as when there is a tidally influenced current. However, this impact will be minor since floodgate closure only takes place for a short period of time (less than 3 to 4 hours). This time period is not all that much longer than the normal 2-hour slack-water period that happens at every high tide. After closure, floodgate opening will occur when tide levels are ebbing, resulting in the release of estuarine water toward the ocean area which has the greatest volume of water and most turbulence. In general, gate closure will take place when the estuary water level is close to 7 feet NGVD. The volume associated with this level in the area of the confluence of the Saugus and Pines Rivers alone (from Route 107 on the Saugus to the B&M railroad bridge on the Pines) is estimated at 3,200 acre-feet while thermal releases of 100 mgd for 4 hours are estimated at 50 acre-feet. If a major storm is expected with heavy runoff and extremely high tides, the floodgate will be closed sooner, in the 2 to 5 feet NGVD range for a period estimated at about 7 hours. The volume associated with this elevation in the area of the confluence of the two rivers is about 1,800 acre-feet, while the release volume from General Electric and RESCO would amount to about 90 acre-feet for the 7-hour closure time. Although the heated water release makes up a larger portion of the total value, there are additional factors to reduce the impacts, namely, there may be large amounts of runoff occurring during closure resulting in a rapid rise in volume, there will generally be high winds which will cause the plumes to mix quickly with the estuarine waters and closures will generally take place in late fall, winter and early spring when ambient water temperatures are cooler. The most important factor to consider, however, is the infrequent occurrence of this condition.

d. Impacts of Rising Sea Level on Water Quality. Any future rise in sea level as discussed in the Hydrology and Hydraulics Appendix will result in an increased frequency of wetland flooding and increased length of time during which the mudflats within the estuary will be underwater. This is true for the natural geometry and for the open floodgate condition. Increased shoreline erosion resulting in increased turbidity will take place until the estuary adjusts to the new tide level. Also, because there is substantially more volume available for storage at higher elevations, there will be an overall increase in flushing volume. The increased flushing volume will result in increased channel velocities and increased suspended solids loadings until the channel geometry has adjusted relative to the change in velocities. (See table 40 of Hydrology and Hydraulics Appendix). If the floodgate is closed for high non-storm spring tides, flushing during these events would be somewhat reduced. However, overall mean tidal flushing would still increase. All the beneficial aspects of increased flushing on water quality would occur during the open gate period with rising sea level including increased oxygen capacity of the tidal prism, increased nutrient exchange, and increased dilution of pollutants. Rising sea level would quickly replace any loss in flushing capability brought about by construction of the Regional Floodgate Plan. The historical sea level increase for the project area has been estimated at nearly 1 foot of rise in 100 years. Others have indicated that up to about 4 feet is possible in the next 100 years. More detailed information is given in the Hydrology and Hydraulics Appendix.

A water quality concern relates to the potential increasing duration and number of gate closures with rising sea level (table 41 of Hydrology and Hydraulics Appendix). Under today's conditions, the gates would be closed about 0.02-0.07 percent of the time resulting in immeasurable water quality changes. With sea level rise of 1,2,3 or 4 feet, the percent of time of gate closure could be 0.8-1.5, 6-10, 18-26, or 33-41, respectively. Resulting water quality conditions in the estuary would be the function of future inflowing water quality which is unknown but should generally improve with continued enforcement of clean water laws. Also, as previously stated, overall tidal flushing under the open gated condition would continue to increase. During gate closures of increasing duration and frequency, some reduction in dispersion and mixing of pollutants and thermal discharges would be expected. As well, some salinity and D.O. reduction may occur. However, the greater mean tidal flushing when the gates are open would rapidly cause mixing, dispersion and removal of pollutants. Some easily settled pollutants may be somewhat retained. It is felt that water quality changes resulting with a foot of sea level rise will be hard to notice. Changes with 2 or more feet of rise would become increasingly more apparent. This would be especially true if frequent prolonged gate closures necessitated the installation of locks and pumps to accommodate navigation passage. In this case, a freshwater body similar to the Mystic or Charles River basins could be created, significantly altering water quality conditions. Salinity would dramatically change, however, the change in other parameters would be a function of pollutional loads at that time, which are unknown. Future detailed studies of all options would be required before any decision is made regarding future installation of locks and pumping. Other options could include low dikes to raise the start of flood damage thereby requiring gate closure a few times a year during coastal storms as would presently occur. Water quality with the low dike option would not be expected to change significantly from today's conditions.

e. Water Quality During Construction. The proposed construction sequence, as described in the design and cost appendix, indicates that at least 5,200 SF of flow area will be maintained below mid-tide level at all times. Similar to the N3 scheme previously discussed, this should have negligible effect on basin tide levels and flushing. No observable changes in water quality would be expected with the exception of some turbidity increase at the construction site. Construction procedures will be conducted in a manner to minimize any temporary increase in turbidity and to limit the introduction of oil and grease from construction materials and equipment.

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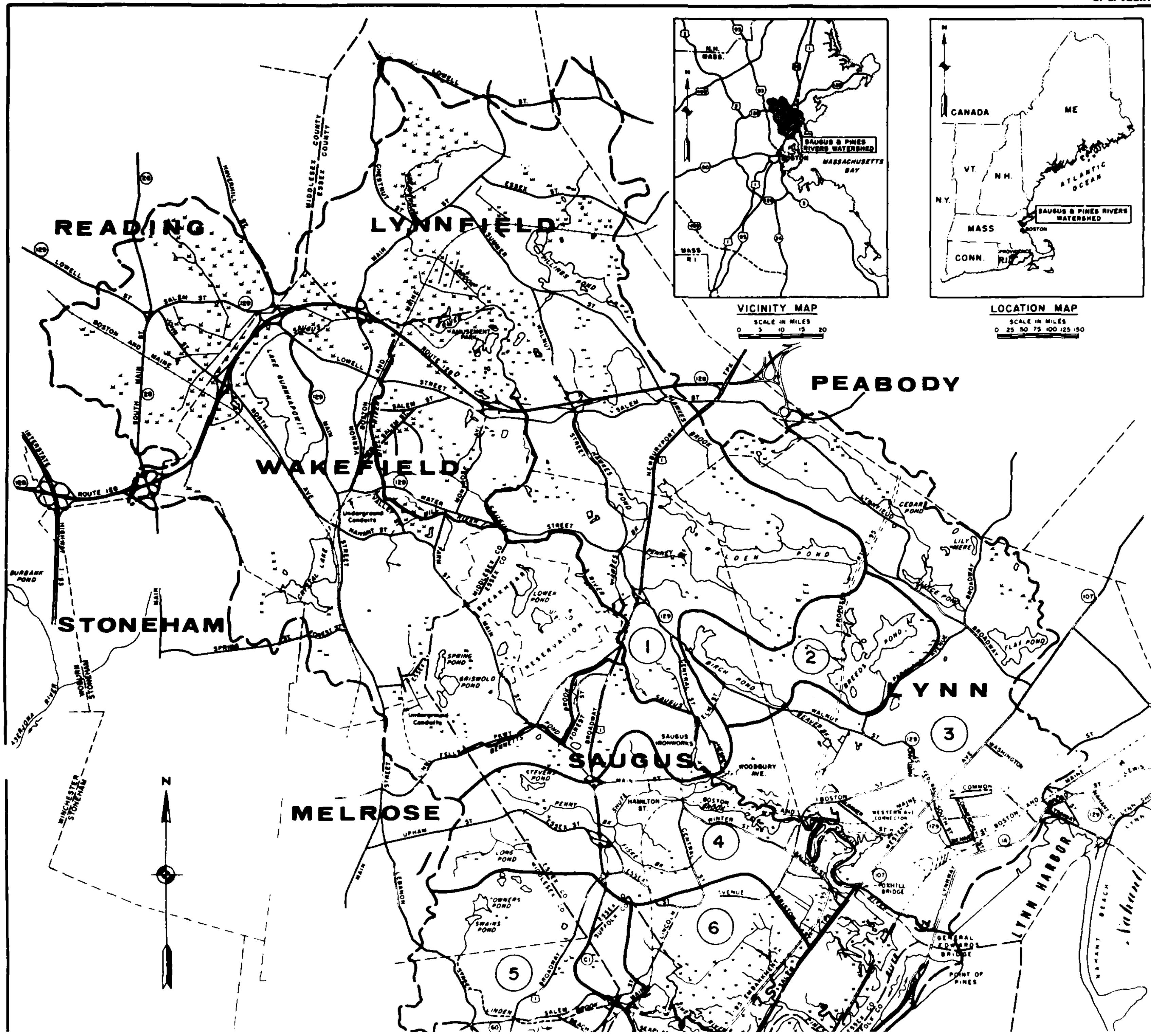
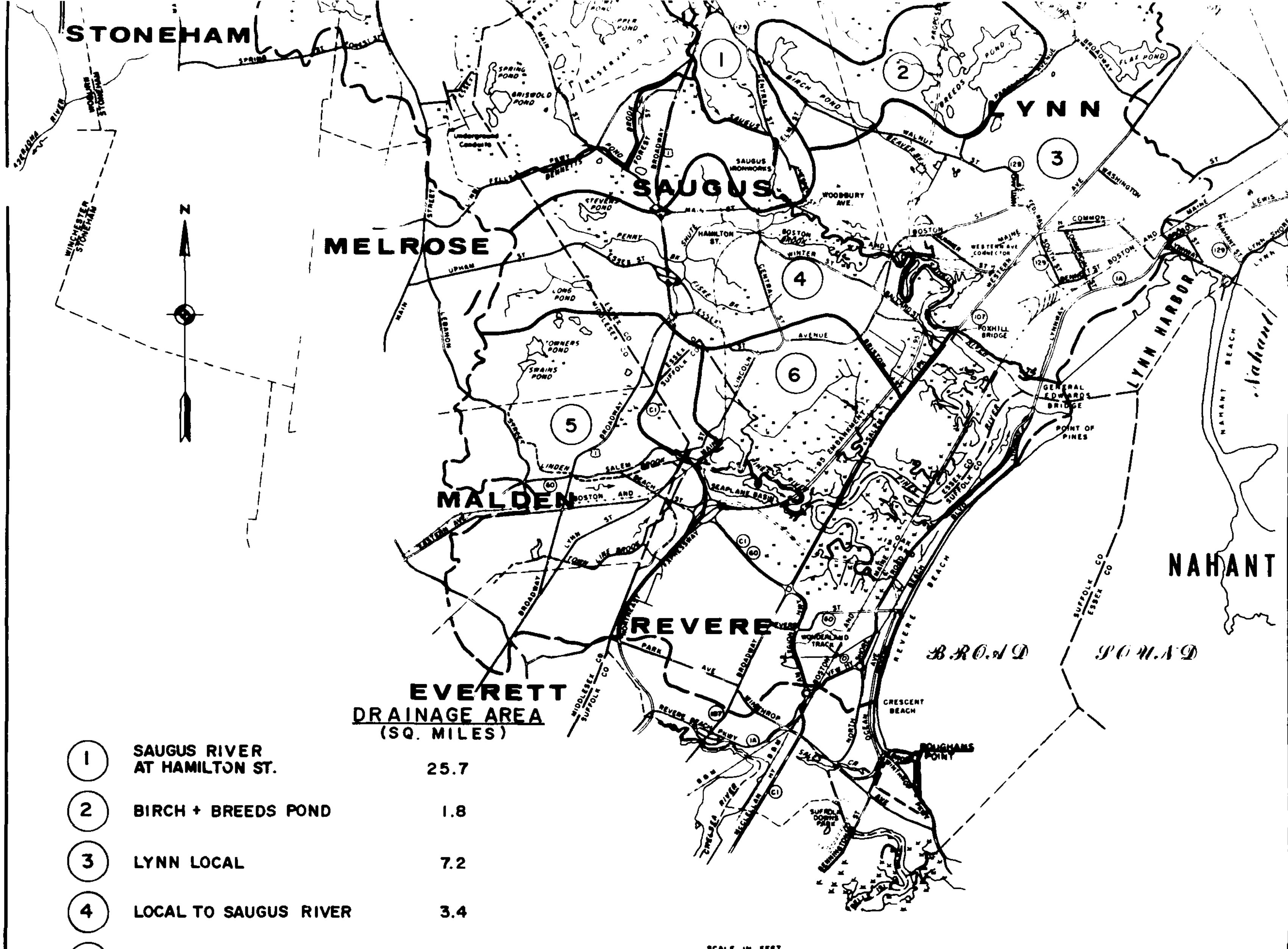
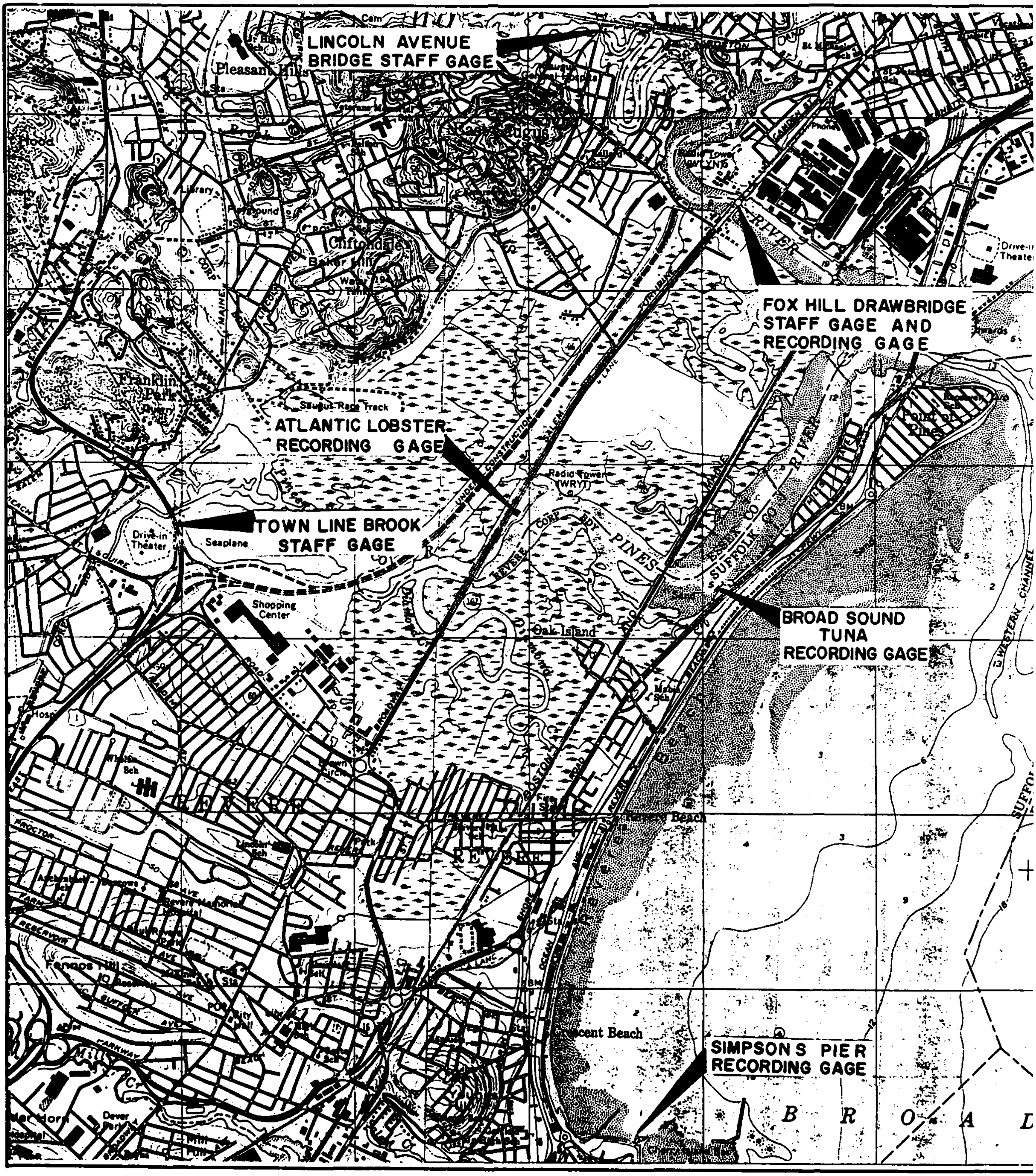

VICINITY MAP
SCALE IN MILES
0 5 10 15 20
SAUGUS & PINES RIVERS WATERSHED
MASSACHUSETTS BAY
BOSTON
LOCATION MAP
SCALE IN MILES
0 25 50 75 100 125 150
CANADA
ME
VT.
N.H.
N.Y.
MASS.
CONN.
R.I.
ATLANTIC OCEAN
SAUGUS & PINES RIVERS WATERSHED
READING
LYNNFIELD
PEABODY
WAKEFIELD
STONEHAM
SAUGUS
MELROSE
LYNN
LYNN HARBOR
Nahant Bay
LAKE QUANNAPOWITT
SAUGUS IRONWORKS
GENERAL EDWARDS BRIDGE
POINT OF PINES
FOXHILL BRIDGE
1
2
3
4
5
6
N



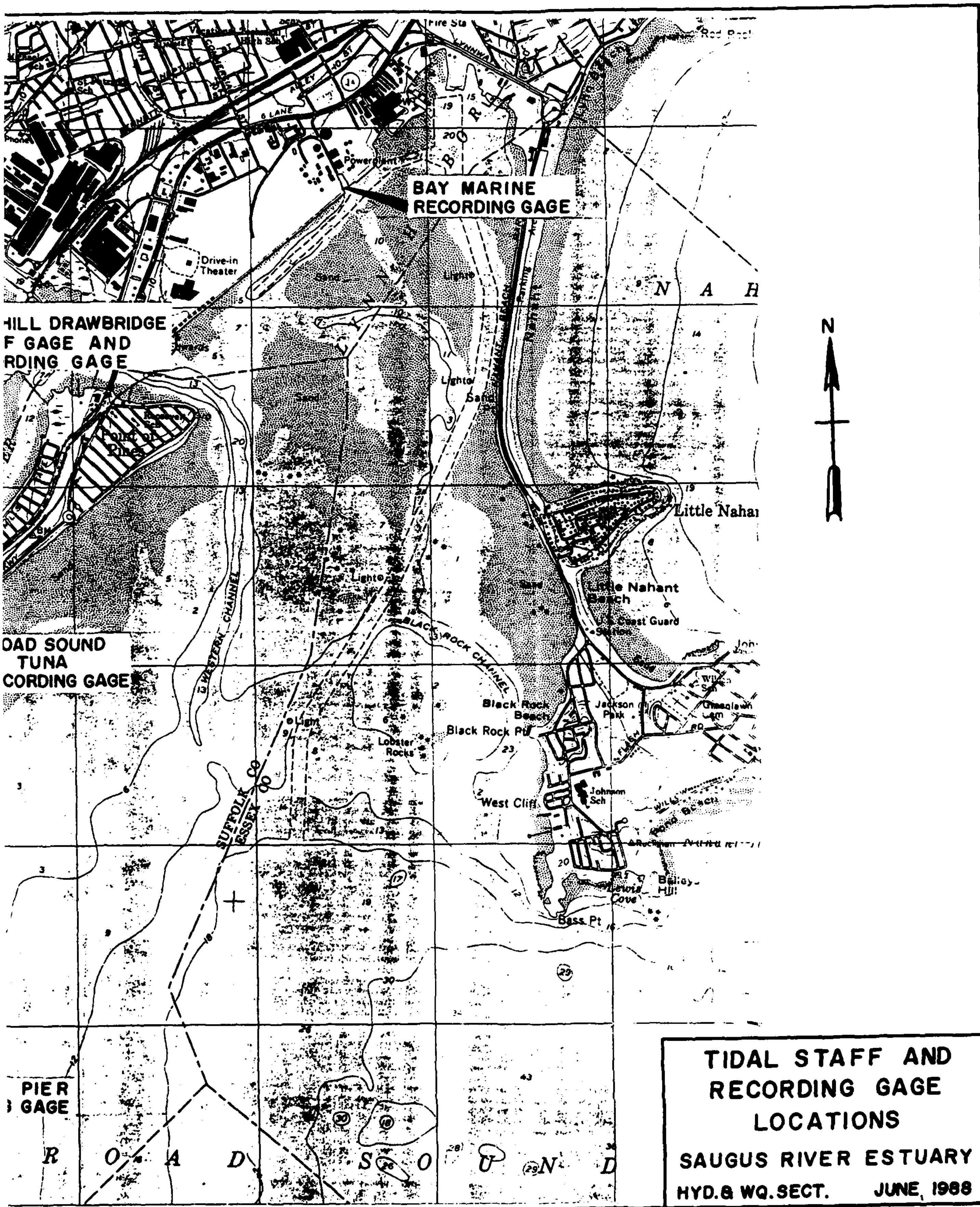
DRAINAGE AREA (SQ. MILES)

1	SAUGUS RIVER AT HAMILTON ST.	25.7
2	BIRCH + BREEDS POND	1.8
3	LYNN LOCAL	7.2
4	LOCAL TO SAUGUS RIVER	3.4
5	LINDEN & TOWN LINE BRKS. (PINES RIVER INFLOW)	4.0
6	PINES RIVER LOCAL	4.9
	TOTAL:	47.0

PLATE I



LINCOLN AVENUE
BRIDGE STAFF GAGE
FOX HILL DRAWBRIDGE
STAFF GAGE AND
RECORDING GAGE
ATLANTIC LOBSTER
RECORDING GAGE
TOWN LINE BROOK
STAFF GAGE
BROAD SOUND
TUNA
RECORDING GAGE
SIMPSON S PIER
RECORDING GAGE
Pleasant Hills
Cliftondale
Franklin Park
Saugus Race Track
Drive-in Theater
Shopping Center
Oak Island
REVERE
Revere Beach
Crescent Beach
B R O A D



TIDAL STAFF AND RECORDING GAGE LOCATIONS

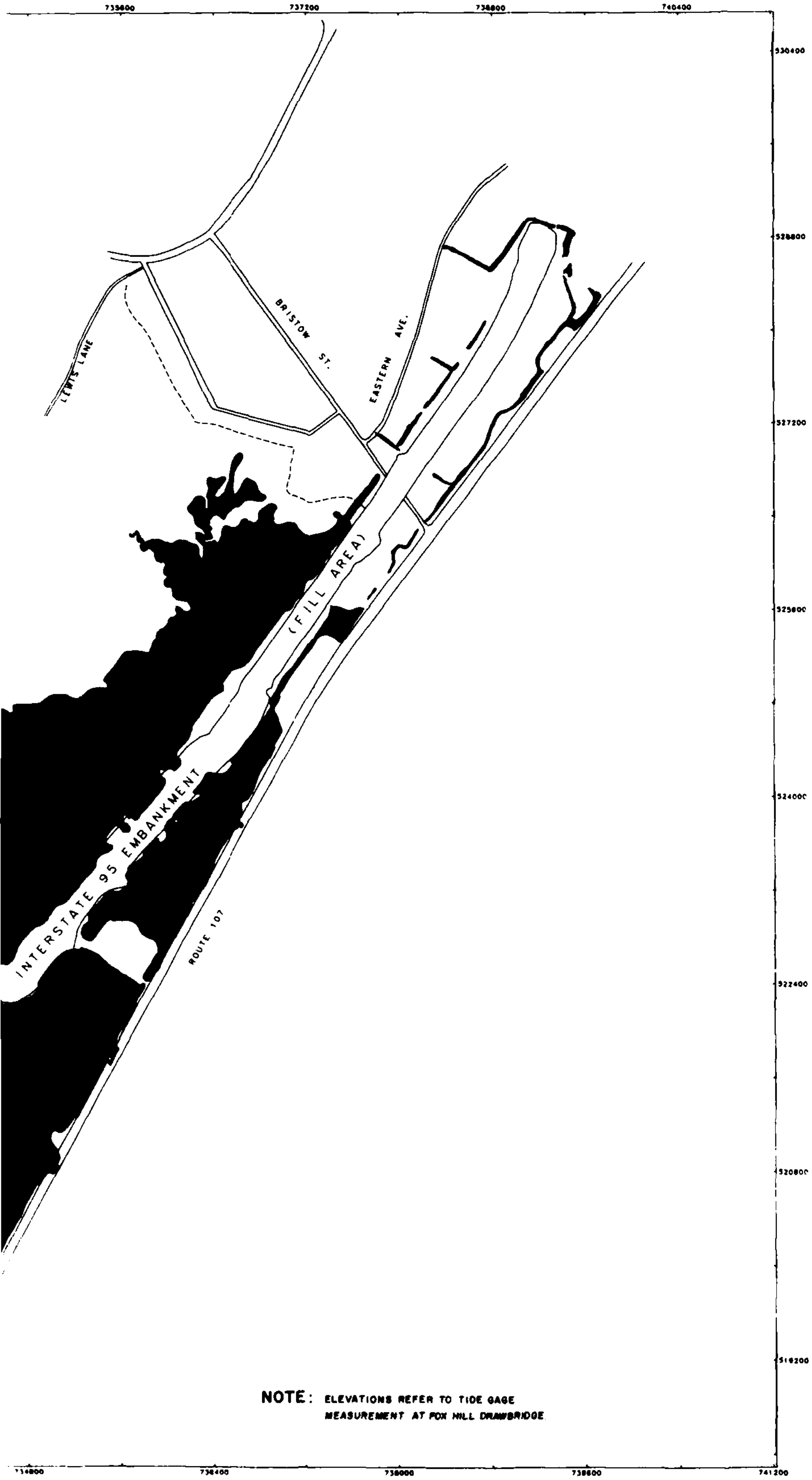
SAUGUS RIVER ESTUARY

HYD. & WQ. SECT. JUNE, 1988

PLATE 2

SUB AREAS A & B TIDAL OVERLA
PINES & SAUGUS RIVERS
OCTOBER 6, 1987

SCALE 1 : 4800
1" = 400 Feet



NOTE: ELEVATIONS REFER TO TIDE GAGE MEASUREMENT AT FOX HILL DRAWBRIDGE

AS A & B TIDAL OVERLAY
P NES & SAUGUS RIVERS
OCTOBER 6, 1987

SCALE 1 : 4800
1" = 400 Feet
0 400 800 1200

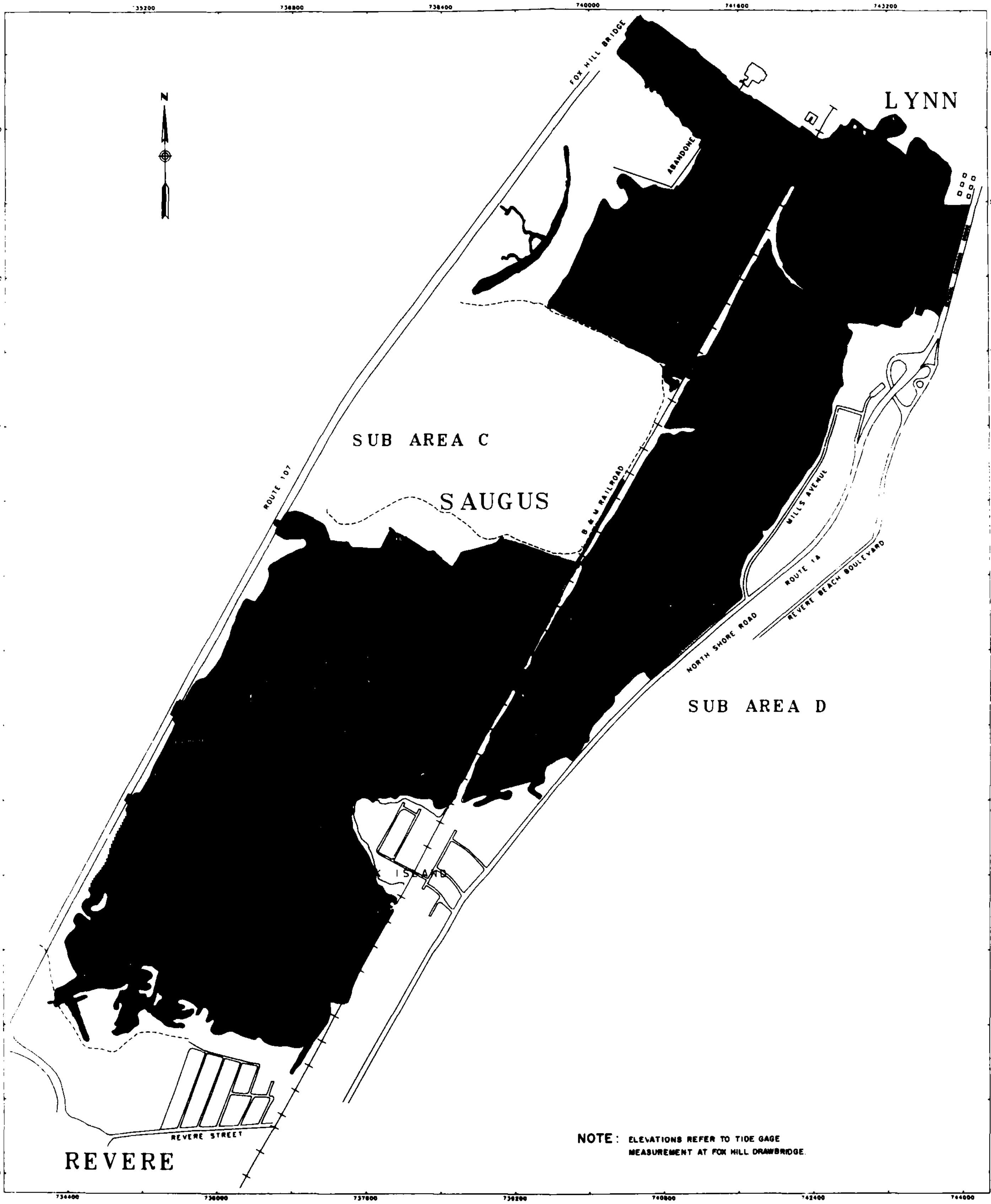
LEGEND

EDGE OF FILLED LAND

PLATE 3A

ESTIMATED WATER SURFACE COVERAGE FROM AERIAL PHOTOGRAPHY

HYD. & WQ. SECT. JUNE 1988



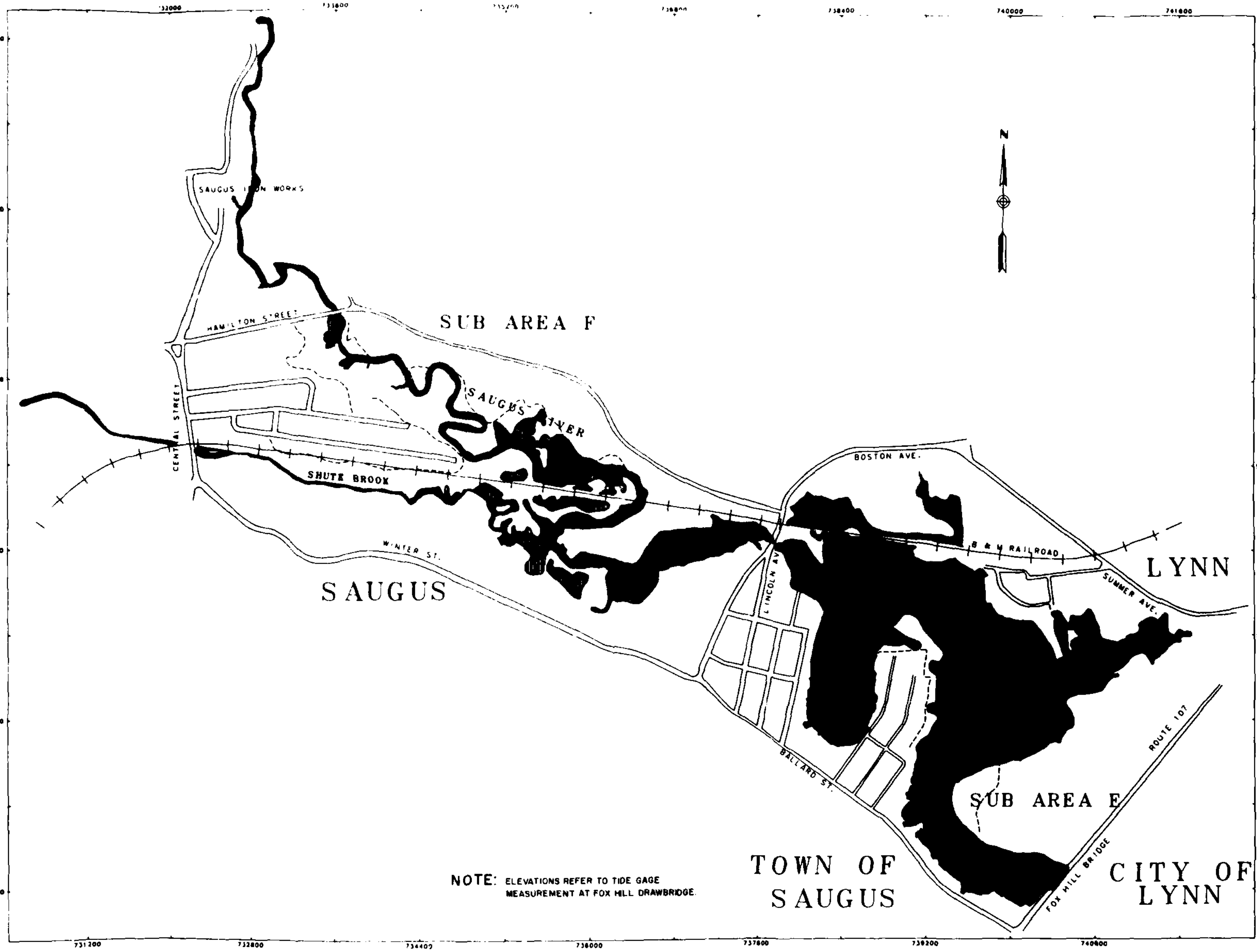
SUB AREAS C & D TIDAL OVERLAY
PINES & SAUGUS RIVERS
OCTOBER 6, 1987

SCALE 1 : 4800
1" = 400 Feet

LEGEND

PLATE 3B
ESTIMATED WATER SURFACE COVERAGE FROM AERIAL PHOTOGRAPHY

HYD. & WQ SECT. JUNE 1988



SUB AREAS E & F TIDAL OVERLAY
PINES & SAUGUS RIVERS
OCTOBER 6, 1987

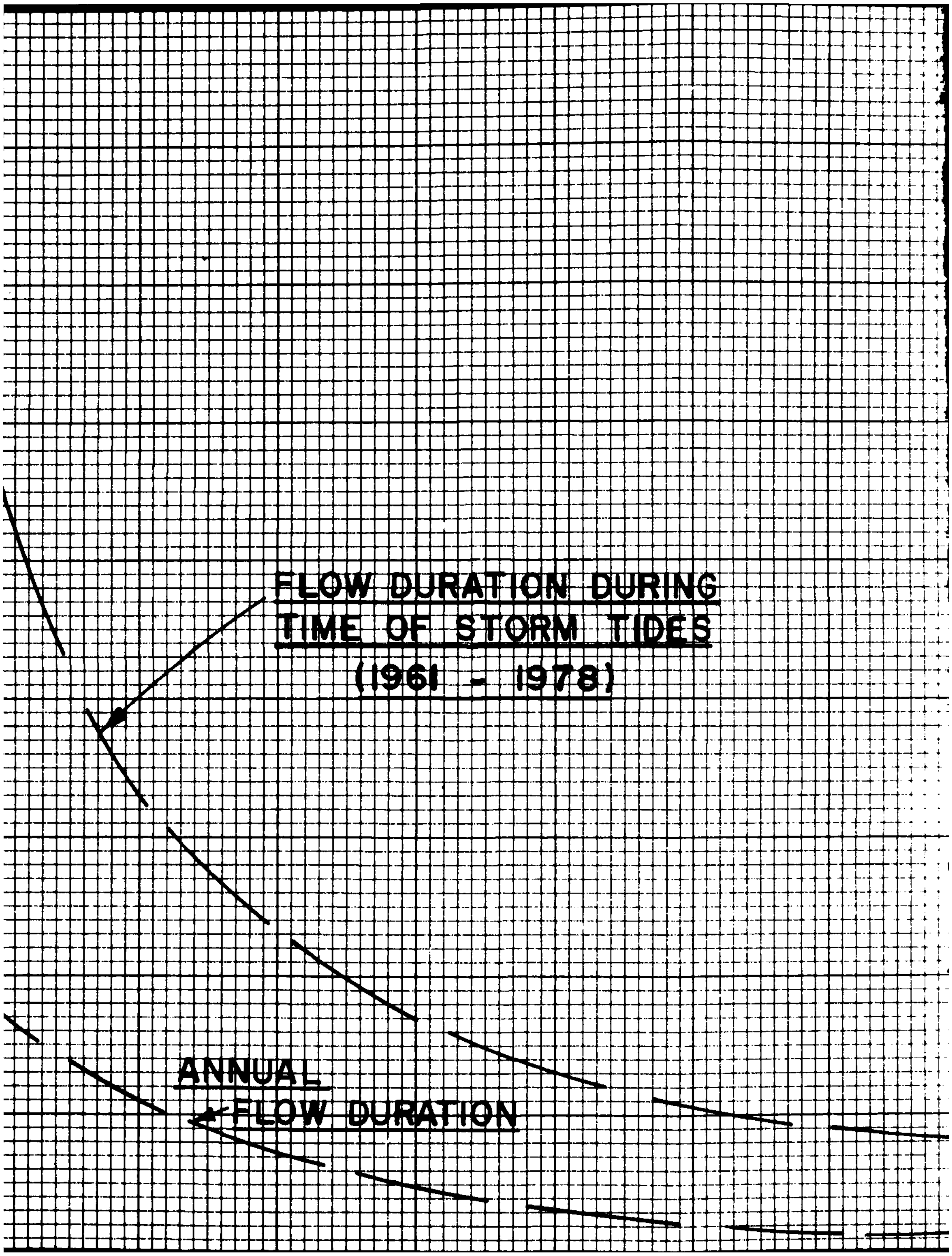
SCALE 1 : 4800
1" = 400 Feet
400 0 400 800 1200

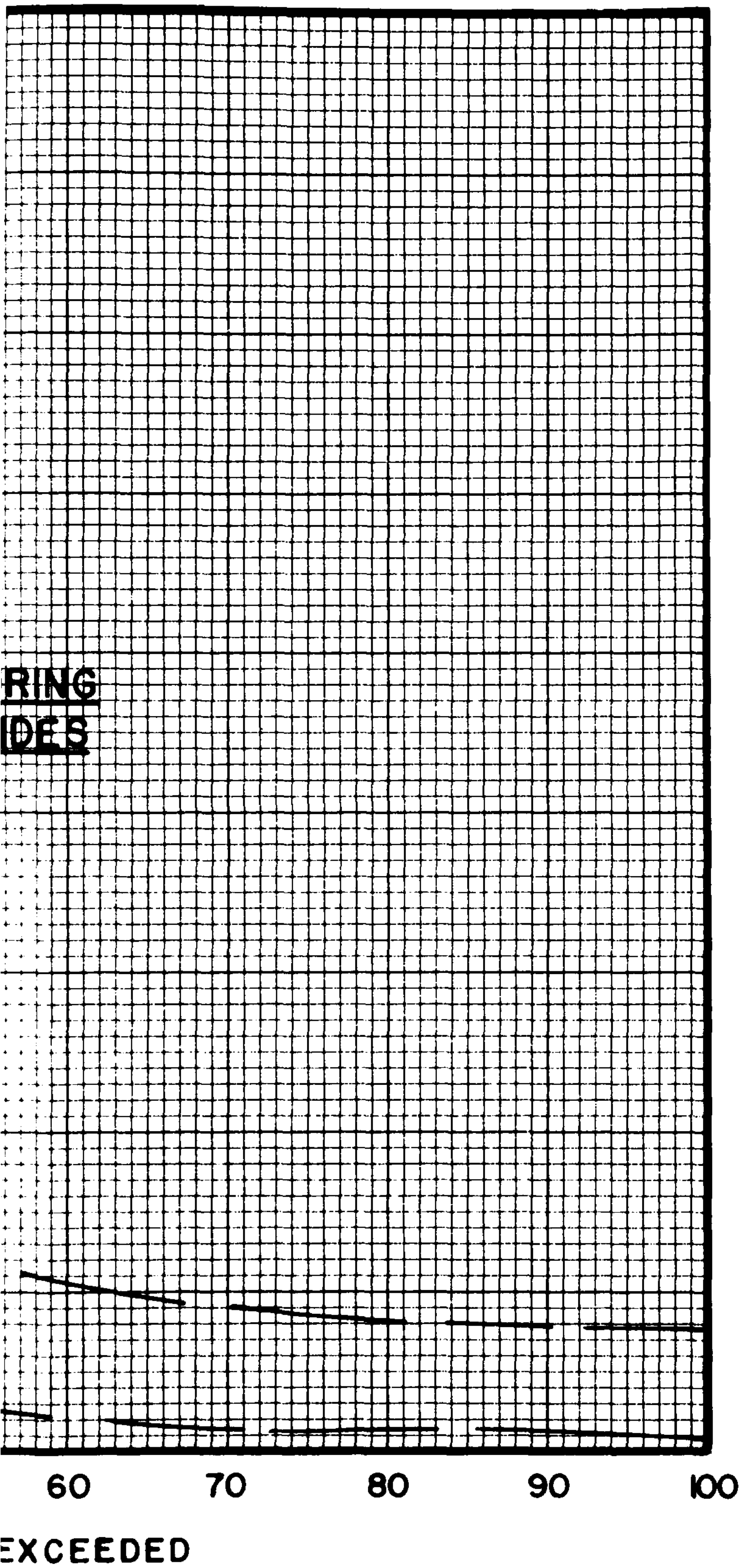
LEGEND

EDGE OF FILLED LAND

PLATE 3C
ESTIMATED WATER SURFACE COVERAGE FROM AERIAL PHOTOGRAPHY

HYD & WQ SECT JUNE 1988


FLOW DURATION DURING
TIME OF STORM TIDES
(1961 - 1978)
ANNUAL
FLOW DURATION



IPSWICH RIVER AT
MIDDLETON, MASS.
(D.A. = 44.5 SQ. MI.)
RUNOFF RATES
DURING STORM TIDES
AT BOSTON

HYDRO. ENG. SECT. JUNE 1988

PLATE 4